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GEOTECHNICAL & GEOLOGIC HAZARD INVESTIGATION (Rev.1) Horizon Neighbourhood Development Summit Powder Mountain Resort Weber County, Utah

IGES Project No. 01628-013

August 3, 2016 (Revised July 19, 2017)

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HORIZON NEIGHBOURHOOD DEVELOPMENT
SUMMIT POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

IGES Project No. 01628-013

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1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical and geologic hazard investigation conducted for the *Horizon Neighbourhood* development located within the Summit Powder Mountain Resort, located near the town of Eden, in Weber County, Utah. Based on the subsurface conditions encountered across the property, it is our opinion that the property is suitable for development provided that the recommendations presented in this report are incorporated into the design and construction of the project.

- The site is overlain with soils ranging in classification from clayey gravel (GC) to lean clay with gravel (CL). The soils across the site are generally susceptible to 'soil creep', a phenomenon whereby under wet conditions the near-surface soil can move down-hill, typically very slowly.
- In general, surficial soils are shallow (typically ranging from 5 to 15 feet, but locally deeper), and overly stable bedrock consisting largely of dolomite, although conglomerate bedrock is also present. Subsurface data suggests that the depth of creeping soils extends to the bedrock/soil interface, although shallower, intermediate creep surfaces are likely to exist locally.
- In consideration of the presence of soil creep, and considering the presence of relatively competent, stable bedrock within the upper ~15 feet, all habitable or critical structures should be founded on a drilled pier foundation. The drilled piers must be firmly embedded into stable bedrock. Recommendations for drilled piers are presented in Section 6.2. Conventional spread footings may be feasible for specific cases, but must be evaluated and approved by IGES on a case-by-case basis (use of conventional spread footings would likely involve significant remedial earth work).
- Although the used of drilled piers to support habitable structures will reduce potential damage to structures over time from soil creep, damage to pavement and/or utilities could still result over time. Use of flexible utility connections could help reduce the impact of ground movement to utilities. The Owner must understand and accept that some maintenance of roads and/or utilities may be necessary over time due to the soil creep.
- Groundwater was not encountered; however, localized spring-like conditions were encountered in some test pits. For some improvements, particularly where basement levels are planned, localized perched groundwater or spring-like conditions may necessitate temporary dewatering during construction. Land drains or other permanent dewatering systems may be desired if problematic local groundwater conditions are encountered during construction.

NOTE: The scope of services provided within this report are limited to the assessment of the subsurface conditions at the subject site. The executive summary is provided solely for purposes of overview and is not intended to replace the report of which it is part and should not be used separately from the report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazard investigation conducted for the *Horizon Neighbourhood* development located within the Summit Powder Mountain Resort, located near the town of Eden, in Weber County, Utah. Based on the subsurface conditions encountered across the property, it is our opinion that the property is suitable for development provided that the recommendations presented in this report are incorporated into the design and construction of the project. The purposes of this investigation were:

- To assess the nature and engineering properties of the subsurface soils across the site;
- To provide recommendations for general site grading and design and construction of foundations, slab-on-grades, exterior concrete flatwork, and roadways; and
- To provide an assessment of geologic hazards that may impact the site.

The scope of work completed for this study included a site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal dated May 6, 2016 and your signed authorization. The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report (Section 7.1).

2.2 PROJECT DESCRIPTION

Our understanding of the project is based primarily on our previous involvement with the Summit Powder Mountain Resort project, which included two geotechnical investigations for the greater 200-acre Summit Powder Mountain Resort expansion project (IGES, 2012a and 2012b) and subsequent geotechnical consulting for several other aspects of the project. The Powder Mountain Resort expansion project is located southeast of SR-158 (Powder Mountain Road), south of previously developed portions of Powder Mountain Resort, in unincorporated Weber County, Utah. The project is accessed by North Powder Ridge Road. The project site is located on what was formerly designated as Lots 19, 20, 22R, and 23R; the site is located south of Summit Pass, and is split into an east-half and west-half by Horizon Run (see *Site Vicinity Map*, Figure A-1 in Appendix A).

We understand that the *Horizon Neighbourhood* development will include 27 assorted types of vacation homes, largely cottage-type structures similar to the nearby *Ridge Nests* development, and associated infrastructure including interior roadways, parking areas, and utilities over an approximately 6.3-acre site. The project will also include a communal structure or lodge.

The site is on a natural slope, draining to the southeast. The slope gradient varies across the site, from a maximum of about 2.7H:1V on the north (former Lot 23R), to a relatively flat 7H:1V on the southeast.

This report has been revised from the original report dated August 3, 2016; Plate A-1 and Plate A-3 have been modified to reflect the most current site plan. All other findings, conclusions, and recommendations remain unchanged.

3.0 METHODS OF STUDY

3.1 LITERATURE REVIEW

A number of pertinent publications were reviewed as part of this investigation. Western Geologic (2012) conducted a reconnaissance-level geologic hazard study for the greater 200-acre Powder Mountain expansion project, including the Horizon Neighbourhood property. The Western Geologic (2012) study modified some of the potential landslide hazard boundaries that had previously been mapped at a regional scale (1:62,500 and 1:100,000, respectively) by Coogan and King (2001) and Elliott and Harty (2010). An updated version of the regional-scale geologic map (Coogan and King, 2016) was also reviewed and compared with the previous version upon which the Western Geologic (2012) study was based. These regional-scale documents were reviewed, in addition to other regional-scale landslide maps produced by Colton (1991; 1:100,000 scale) and Giraud and Shaw (2007; 1:500,000 scale) and liquefaction maps produced by Anderson et al. (1994; 1:48,000 scale) and Christensen and Shaw (2008, 1:250,000 scale). Sorensen and Crittenden, Jr. (1979) provides 1:24,000 scale geologic mapping of the Huntsville Quadrangle, which documents the surficial geology of the Horizon Neighbourhood project area at a more local scale. The corresponding United States Geological Survey (USGS) topographic map for the Huntsville Quadrangle provides physiographic and hydrologic data for the project area. A single Federal Emergency Management Agency (FEMA) flood map (effective in 2015) that covers the project area was also reviewed. The Weber County Special Study Area maps were reviewed for the project area. The Quaternary Fault and Fold Database (USGS and Utah Geological Survey (UGS), 2006), was reviewed to identify the location of proximal faults that have had associated Quaternary-aged displacement. The two geotechnical investigations for the Powder Mountain property performed by IGES (2012a, 2012b) were reviewed in detail to provide an understanding of the nature of the subsurface materials at the site and to assist in the geologic mapping of the site.

3.2 FIELD INVESTIGATION

The field exploration program included site reconnaissance and field mapping, and two rounds of subsurface exploration. The initial field exploration program began on June 8, 2016 and was completed on June 9, 2016. Eight (8) exploration test pits were excavated to depths generally ranging from 12 to 15 feet below existing grade. The exploration test pits were excavated with the aid of a Caterpillar 320E tracked excavator. Refusal on hard bedrock was encountered in two of the test pits (TP-2, TP-8).

As a result of unanticipated subsurface conditions, including evidence of excessive soil creep, observed in the first round of exploration test pits, a second phase of subsurface exploration was conducted between July 6 and 7, 2016. Four (4) additional test pits and three (3) potholes were excavated with the aid of a Caterpillar 345C tracked excavator to provide supplementary subsurface data across the property. The potholes were simply deep, steep holes dug with the intent to identify the top of bedrock at depth, and were not logged and sampled in the same manner as the test pits. Refusal on hard bedrock was encountered in one of the supplemental test pits (TP-11); the remaining supplemental test pits were excavated to depths generally ranging from 12 to 15 feet below existing grade.

The *Geotechnical Map,* Plate A-1 in Appendix A, shows the approximate location of the exploration test pits and potholes and the surficial geologic materials as mapped from the site reconnaissance. The exploration test pits and potholes were specifically located to assess the presence or absence of adverse geologic features, assesse the depth to bedrock, and to observe subsurface conditions. Subsurface conditions as encountered in the exploration test pits and potholes were logged at the time of our investigation by a licensed geologist. The test pit logs are presented in Figures A-2 through A-13 of Appendix A. A *Key to Soil Symbols and Terminology* is presented as Figure A-14 and a *Key to Physical Rock Properties* is presented as Figure A-15.

Bulk soil samples were obtained from the test pit explorations; due to the coarse nature of the subsurface materials, few ‘undisturbed’ tube samples were able to be collected. All soil samples were transported to our laboratory for testing to evaluate the engineering properties of the earth materials observed.

3.3 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- In situ moisture content & unit weight (ASTM D7263 and D2216)
- Atterberg Limits (ASTM D4318)
- Fines Content (% passing the #200 sieve) (ASTM D1140)
- Gradation (ASTM D6913)
- Direct Shear Test (ASTM D3080)
- Ring Shear Test

Results of the laboratory testing are included with this report in Appendix B.

3.4 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

The site is in a relatively natural state, and is nearly entirely covered in vegetation. Native shrubs and grasses cover most of the property, while thick stands of quaking aspen trees are found in some areas. Horizon Run, which passes southwest to northeast, effectively bisects the property into western and eastern halves. The western half of the property contains the steepest topography, with slopes as much as 37 percent (2.7H:1V). Topography on the eastern half of the property is much more subdued, with slopes averaging around 14 percent (7H:1V). Slopes become significantly steeper (34 percent; 2.9H:1V) on the eastern half of the property south of the southernmost proposed unit, however. The elevation across the site ranges from approximately 8,842 feet in the northwestern corner to approximately 8,588 feet (msl) in the southernmost corner.

At the time of the fieldwork, the easternmost portion of the eastern half of the property was in the process of undergoing earthwork to create a ski run associated with a nearby bridge, in which a large mound of native materials had been piled and was in the process of being leveled off.

4.2 SUBSURFACE CONDITIONS

The subsurface soils were investigated by excavating a total of twelve (12) exploration test pits and three (3) potholes at representative locations across the site. Generally, the depth of the exploration test pits ranged from 12 to 15 feet, and refusal on hard bedrock was encountered in three test pits (TP-2, TP-8, and TP-11). The locations of the test pits are illustrated on Plate A-1, *Geotechnical Map*; detailed test pit logs are presented in Figures A-2 through A-13. The earth materials encountered in the exploration test pits were visually classified and logged by an IGES licensed geologist. The subsurface conditions encountered during our investigation are discussed below.

4.2.1 Earth Materials

Based on our observations, the site is generally covered by a veneer of topsoil ranging in depth from 6 inches to 24 inches. The topsoil is generally underlain by bouldery colluvium derived from weathered Wasatch Formation, which in turn is underlain by disaggregated Wasatch Formation bedrock. The Wasatch Formation was found to be underlain by hard bedrock of the Nounan Dolomite, though clayey and highly weathered dolomite units were often encountered between the Wasatch Formation and Nounan Dolomite bedrock. Undocumented fill material was not observed in the test pits. Descriptions of the geologic units encountered are presented in the following paragraphs.

<u>Artificial Fill</u>: Artificial fill (denoted as either Af or Afc on Plate A-1) was observed throughout the site. Af represents asphalt and road embankment fill material found in association with and restricted to the existing roads that bound the property, including North Powder Ridge Road and

Horizon Run. Afc consisted of a large stockpile of what appeared to be native soils, brought in to construct a new ski run associated with the nearby bridge over North Powder Ridge Road.

Topsoil: Generally consists of dark brown to grayish brown to brownish black lean clay with gravel. The soil is generally loose to slightly cohesive, and exhibits low plasticity. The unit is typically slightly moist to dry, and contains abundant plant and tree roots throughout. Subrounded to subangular quartzite rock fragments were found to comprise as much as 50% of the unit, with some boulders as much as 2 feet in diameter. The topsoil unit was encountered in all of the exploration test pits and potholes, and was found to be between 6 inches and 2 feet thick.

Colluvium: Two classes of colluvial units were observed to underlie the topsoil and be the source material for the topsoil (Qcl and Qcc/Qls on Plate A-1). The most prevalent form was a dark brown loosely consolidated unit (map symbol Qcl) that was gradational between lean clay with gravel (CL) and clayey gravel (GC). This form was often silty, poorly sorted, and contained subrounded to subangular quartzite rock fragments that comprised between 15 and 50% of the unit. Boulders were found to be as much as 2 feet in diameter. The second, less common form of colluvium was a light brown, well-cemented, silty clay with gravel (CL-ML) gradational to a silty, clayey gravel (GC, GM) (map symbol Qcc/Qls). This form contained abundant pinhole voids throughout, was poorly sorted, and contained subrounded to subangular quartzite rock fragments that comprised between 25 and 50% of the unit. Boulders were found to be as much as 1.5 feet in diameter, and were on average smaller than the loose colluvium unit. It is possible that this cemented unit is derived, at least in part, from the Qcl unit that was remobilized in a small landslide associated with the small headscarp found between TP-1 and TP-2.

Bedrock Colluvium: This material (denoted as Qcb on Plate A-1) consists of colluvium who's parent material largely consists of dolomite; this material was found to be present along the northeastern part of the Horizon Run roadcut and along the northern road cut of North Powder Ridge Road. It consisted of clasts of dolomite bedrock generally less than 6 inches in diameter that was not associated with any in-place bedrock outcrop.

Wasatch Formation: The Wasatch Formation (map symbol Tw on Plate A-1) consists of conglomerate bedrock that readily disaggregates to dark to moderate reddish brown sandy clay with gravel (CL) gradational to clayey sand with gravel (SC). The unit contains abundant subrounded to subangular quartzite rock fragments comprising between 15 and 75% of the unit. Cobbles were generally less than 1 foot in diameter, with a mode average size of 2 to 4 inches. Pinhole voids were commonly encountered in places, and the unit contained some thin (<3 inches thick) silt and clay lenses. Thin (<4 inches thick) black paleosols were also encountered within the unit in two test pits (TP-11 and TP-5).

Transitional Units: Between the Wasatch Formation and the underlying Nounan Dolomite, several transitional units were typically encountered. These included dark red to dark yellowish orange fat clays that could represent pre-Tertiary age paleosols, and dark gray to dark yellowish orange sandy fat clays that consist of decomposed bedrock. The paleosol units were commonly found to contain slickensides, which were observed both naturally and generated through excavation of the unit with a pick. These also commonly contained pinhole voids and minor (<5%) amounts of small (<6 inches in diameter) angular dark gray dolomite bedrock clasts, though quartzite clasts were also observed in places. The decomposed bedrock units generally exhibited relict bedrock bedding and structure, as well as variable-sized angular dolomite bedrock clasts throughout that were soft, sandy, and friable.

Nounan Dolomite: The Nounan Dolomite (map symbol Cn on Plate A-1) is a medium gray to dark gray, finely sparry dolomite bedrock that met with refusal in several of the test pits. Where less weathered, the unit exhibited well-developed blocky jointing and thin bedding. In general, the unit exhibited a highly variable degree of weathering, and could be soft and easy to break with hands or very hard to break with repeated blows with a rock hammer. Typically, the harder bedrock was overlain by several feet of soft, sandy, highly weathered bedrock (see *Transitional Units* above), and in some cases was interbedded with fat clay beds with a relict shaley structure (see TP-11).

Pleistocene Landslide: This material (denoted as Qlso on Plate A-1) was found as a lobe extending to the south (downslope) from a noted landslide headscarp in the southernmost part of the property. It was characterized by generally irregular, slightly hummocky topography and was largely devoid of trees within its trace. This unit is part of the previously-mapped landslide deposit that crosses the Horizon Neighbourhood property (Sorensen and Crittenden, Jr. (1979) and Western Geologic (2012); see Section 5.1.2).

Anomalous Units: In addition to the commonly observed units described above, a couple additional units were locally or anomalously encountered in some of the excavations. One of these was a pale yellowish orange to dark yellowish orange fat clay that may represent a localized pond clay within the uppermost portion of the Wasatch Formation. This unit did not exhibit slickensides, and was encountered overlying the conglomeratic Wasatch Formation in TP-4, TP-5, and TP-7. A second unit that was locally encountered was what appears to be alluvial deposits and/or vug infilling within the transitional units in TP-9, TP-11, TP-12, and PH-3. These units were typically dark reddish brown to dark yellowish orange, and were clayey sands gradational to sandy fat clays that contained occasional clasts of both Wasatch Formation quartzite and dolomite bedrock. Pinhole voids were also commonly found in these alluvial units.

The lines shown on the enclosed logs and plates represent the approximate boundary between the different earth materials. Due to differing depositional natures of natural earth materials, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

4.2.2 Groundwater

Groundwater was encountered in three of the excavations (TP-3, TP-6, and TP-8). The most notable groundwater occurrence was in TP-6, in which groundwater had filled the test pit with a water column thickness of 2.2 feet within a couple hours of initial excavation (water was 2.2 feet deep measured from the bottom of the test pit). In this test pit, the groundwater was entering the pit at a depth of approximately 7 feet below the existing ground surface from the northern and eastern walls. Given the rapid accumulation of groundwater into the test pit, this depth may reflect the water table level at this location. TP-3 encountered groundwater slowly seeping into the northeastern corner of the test pit at a depth of approximately 9 feet below the existing ground surface, and TP-8 encountered groundwater slowly seeping through the northern wall of the test pit at a depth of approximately 5 feet below ground surface. Measurable accumulation of groundwater was not experienced in either TP-3 or TP-8, and the seepage depths do not reflect the level of the water table.

Due to the season of our investigations (late spring and early summer), we anticipate groundwater levels to be just below their seasonal high. It is our experience that during snowmelt, runoff, irrigation on the property and surrounding properties, high precipitation events, and other activities that the groundwater level can rise several feet. Fluctuations in the groundwater level should be expected over time.

4.2.3 Strength of Earth Materials

Two consolidated-drained direct shear tests were completed under drained conditions on relatively undisturbed samples (tube samples) obtained from the prevailing surficial clayey soils. The samples were obtained from different locations to provide reasonable coverage across the site and to provide a basis for assessing representative strength parameters for geotechnical analysis such as slope stability. The test results are summarized in the following table:

Table 4.2.3
Summary of Direct Shear Test Results

Test Pit	Depth (ft.)	Friction Angle (deg.)	Cohesion (psf)	Notes
T-02	5	36	160	LL=60, 82% fines - CH
T-12	6	44	0	CL with sand

The values presented in Table 4.2.3 are peak values.

A three-point ring shear test is currently being conducted on a sample of clayey soil obtained from an apparent shear zone identified in TP-1. The test is on-going; however, based on preliminary results completed for a single point, the values obtained thus far is a secant peak friction angle of 11.6 degrees and a secant residual friction angle of 9.1 degrees. The preliminary test results suggest that the clayey soils are fairly weak when in a residual state, which likely accounts for the 'soil creep' observed throughout the site.

4.3 STABILITY OF NATURAL SLOPES

4.3.1 Slope Stability

The stability of the existing natural slopes have been assessed in accordance with methodologies set forth in Blake et al. 2002 and AASHTO LRFD for Bridge Design Specifications with respect to Sections A-A' and B-B', illustrated on Plate A-1. The stability of the slopes were modeled using SLIDE, a computer application incorporating (among others) Spencer's Method of analysis. Calculations for stability were developed by searching for the minimum factor of safety for a translational-type failure. Homogeneous earth materials and arcuate failure surfaces were assumed. Analysis was performed for the following cases:

a) Static analysis of proposed geometry
b) Static analysis with transient high groundwater
c) Yield acceleration of proposed geometry (for slope deformation analysis), and
d) Pseudo-static analysis of proposed geometry

Pseudo-static (seismic screening) analysis of the proposed slope was performed in general conformance with Blake et al. 2002, ASCE 7-10 and AASHTO LRFD for Bridge Design Specifications. The design seismic event was taken as the ground motion with a 2 percent probability of exceedance in 50 years (2PE50). Based on information provided on the USGS website ground motion calculator, the Peak Ground Acceleration (PGA) associated with a 2PE50 event is estimated to be 0.33g. Half of the PGA, (0.17g), was taken as the horizontal seismic coefficient (k_h) (Hynes and Franklin, 1984), and used in the pseudo-static seismic screen analysis. The results of the analyses have been summarized in Table 4.3.1.

Where the pseudo-static screen analysis of the cross sections resulted in a factor of safety less than one, a simplified Newmark-type displacement analysis was performed in accordance with Bray and Travasarou (2007). The purpose of this additional analysis is to estimate the potential magnitude of seismic slope movement. It is important to note that developers of this simplified approach to estimate displacement consider the results of these analyses to be indices of expected seismic performance and not predications of exact amount or location of slope displacement amount. The results of the analyses have been summarized in Table 4.3.1.

Table 4.3.1
Results of Slope Stability Analyses

Section	Static Factor of Safety	Pseudo-Static Factor of Safety	Yield Acceleration (g)	Estimated Displacement* (inches)
Section A-A’ Upslope	1.2	<1	0.05	12 to 18
Section A-A’ Downslope of Proposed Improvements	1.4	<1	0.1	6 to 12
Section A-A’ within Improvements	1.8	1.0	N/A	N/A
Section B-B’ without Keyway	1.1	<1	0.02	>24
Section B-B’ with Keyway	1.7	1.0	N/A	N/A

*Estimated using methods proposed by Bray and Travasarou (2007)

The results of these analyses indicate that seismic displacement could be between 1 to 2 feet within loose colluvium material (Qcl) and Pleistocene landslide material ($Qlso_2$). The areas identified as being susceptible to this movement are illustrated in the results attached in Appendix C of this report.

Groundwater was generally not encountered during our investigation, although within two test pits water was encountered, presumed to be localized spring-like conditions associated with spring run-off. Our surface reconnaissance did not reveal any obvious signs of near-surface groundwater (e.g., seeps, springs, reeds or heavily-vegetated areas, surficial slumping, etc.). Groundwater data for the site is very limited; however, based on our understanding of the geology and hydrology of the area, groundwater (regional piezometric surface) is not expected to impact the site, although localized areas of perched groundwater or spring-like conditions could impact construction. Groundwater was considered within the analysis of Section A-A’ to identify the critical surface under static conditions (this would model transient spring-like conditions during primary snow melt – if it should occur, it would likely be a localized phenomenon). The water table was modeled to be approximately 5 feet above the boundary of the loose colluvium (Qcl) and the Nounan dolomite bedrock (Cn). The results of the analysis suggest a factor of safety of approximately 1.0. This analysis suggests that ‘soil creep’ that has been documented in this area is associated with seasonal periods of rapid snow melt and temporary high moisture content (‘soil creep’ is described in Section 5.2.1 of this report).

The results of the stability analyses and the slope deformation analysis are presented in Appendix C.

4.3.2 Surficial Stability

Our subsurface investigation indicates that the near-surface soils generally grade from clay with gravel (CL) to clayey gravel (GC). Material identified as 'topsoil' (A/B Horizon) generally ranges in thickness from 1 to 2 feet; the topsoil has developed on the prevailing colluvial cover, and therefore also consists of clayey gravel grading to gravelly clay, but with a higher organic component (abundant roots).

IGES assessed the potential for the upper four feet to become mobilized under saturated parallel seepage conditions. Our assessment assumes four feet of coarse clayey colluvium, fully saturated, and a 2.7H:1V slope. Our model assumes an effective friction angle of 36 degrees and a cohesion of 150 psf, and a saturated unit weight of 135 pcf. Based on this model, a factor-of-safety of 1.91 results. Sample calculations are presented in Appendix C.

Our calculations do not take into account the beneficial effects of plant roots, which were commonly observed throughout the topsoil units. Many of the existing natural slopes are thickly vegetated, which is expected to reduce the likelihood of shallow surficial slope instability.

Based on our infinite slope model, and the foregoing discussion, IGES considers the potential for surficial slope instability on this site to be low.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

5.1.1 Regional Geology

The Summit Horizon Neighbourhood property is located in the western portion of the northern Wasatch Mountains, which have a complex geologic history. The Wasatch Mountains contain a broad depositional history of thick Precambrian and Paleozoic sediments that have been subsequently modified by various tectonic episodes that have included thrusting, folding, intrusion, and volcanics, as well as scouring by glacial and fluvial processes (Stokes, 1987). The uplift of the Wasatch Mountains occurred relatively recently during the Late Tertiary Period (Miocene Epoch) between 12 and 17 million years ago (Milligan, 2000). Since uplift, the Wasatch Front has seen substantial modification due to such occurrences as movement along the Wasatch Fault and associated spurs, the development of the numerous canyons that empty into the current Salt Lake Valley and Utah Valley and their associated alluvial fans, erosion and deposition from Lake Bonneville, and localized mass movement events (Hintze, 1988).

The Wasatch Mountains, as part of the Middle Rocky Mountains Province (Milligan, 2000), were uplifted as a fault block along the Wasatch Fault (Hintze, 1988). The Wasatch Fault and its associated segments are part of an approximately 230-mile long zone of active normal faulting referred to as the Wasatch Fault Zone (WFZ), which has well-documented evidence of late Pleistocene and Holocene (though not historic) movement (Lund, 1990; Hintze, 1988). The faults associated with the WFZ are all normal faults, exhibiting block movement down to the west of the fault and up to the east. The WFZ is contained within a greater area of active seismic activity known as the Intermountain Seismic Belt (ISB), which runs approximately north-south from northwestern Montana, along the Wasatch Front of Utah, through southern Nevada, and into northern Arizona. In terms of earthquake risk and potential associated damage, the ISB ranks only second in North America to the San Andreas Fault Zone in California (Stokes, 1987).

The WFZ consists of a series of ten segments of the Wasatch Fault that each display different characteristics and past movement, and are believed to have movement independent of one another (UGS, 1996). The Summit Horizon Neighbourhood property is located approximately 9.5 miles to the east of the Weber Segment of the Wasatch Fault, which is the closest documented Holocene-aged (active) fault to the property and trends north-south along the Wasatch Front (USGS and UGS, 2006).

The property is underlain by Cambrian bedrock which comprise the upper plate of the Willard Thrust (Sorensen and Crittenden, Jr., 1979), and comprise an allocthonous[1] block of rock that has

[1] Allocthonous: Formed or produced elsewhere than in its present place; of foreign origin, or introduced. (AGI, 2011)

been transported eastward to its present location from the Cordilleran geosyncline[2] (Stokes, 1987). The Willard Thrust is believed to connect and be structurally continuous with the Charleston-Nebo Thrust, which passes through the Salt Lake Valley and beneath Strawberry Reservoir, with the two thrusts connecting near Antelope Island (Stokes, 1987).

5.1.2 Local Geology

Several extant geologic maps cover the Summit Horizon Neighbourhood property. Sorensen and Crittenden, Jr. (1979) provides the most detailed mapping of the general geology of the area, and serves as the base map for the *Regional Geologic Map 1* shown in Figure A-16a and corresponding map legend in Figures A-16b and A-16c. According to Sorensen and Crittenden, Jr. (1979), the property is largely underlain by the undivided Tertiary/Cretaceous Wasatch and Evanston Formations, which underlie the entire western half of the property, and the approximately northern half of the eastern half of the property. A Holocene-aged landslide deposit is mapped across much of the approximately southern half of the eastern half of the property, and the southernmost part of the eastern half of the property is mapped as undifferentiated Holocene colluvium, slopewash, and landslide deposits.

Following upon Sorensen and Crittenden, Jr. (1979), these bodies of mass-movement deposits had their contacts further delineated by Coogan and King (2001; 2016) and Western Geologic (2012) in subsequent mapping efforts. Being a regional-scale map, Coogan and King (2001) lumped the Holocene-aged landslide deposit together with the undifferentiated mass movement deposits, and described these deposits as: “Mass-movement deposits, undivided – Includes slides, slumps, and flows, as well as colluvium, talus, and alluvial fans that are mostly debris flows; composition depends on local sources.” Drawing upon Coogan and King (2001), Western Geologic (2012) kept the same undifferentiated mass movement outline as Coogan and King (2001), but separated out the Holocene landslide of Sorensen and Crittenden, Jr. (1979) that overlies the southern part of the Horizon Neighbourhood property, though with a similar, but slightly different outline and a Late Pleistocene to Holocene age (see *Regional Geology Map 2*, Figure A-17). Finally, Coogan and King (2016) updated their 2001 map by including a similar area to that mapped by Western Geologic as the landslide deposit on the Horizon Neighbourhood property, though it was mapped as Holocene and Pleistocene-aged undifferentiated landslide and colluvial deposits.

No faults have been mapped within 1 mile of the property, and no faults, either active or inactive, have been mapped on or projecting towards the property. An active fault is defined by the Weber County Code of Ordinances as “a fault displaying evidence of greater than four inches of displacement along one or more of its traces during Holocene time (about 11,000 years ago to the present).” (Weber County, 2015)

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[2] Geosyncline: As originally defined, a mobile downwarping of the crust of the Earth, either elongate or basinlike, measured in scores of kilometers, in which sedimentary and volcanic rocks accumulate to the thicknesses of thousands of meters. (AGI, 2005)

Site reconnaissance and geologic mapping of the property was performed as part of the fieldwork for this project, and served largely as the basis upon which the test pit locations were determined. Plate A-1 displays the geologic map produced as part of this mapping effort, and two representative geologic cross-sections are displayed in Plate A-2. A small vegetation-free scar representing the scarp from a recent shallow landslide was noted in the northwestern part of the property (illustrated on Plate A-1). There was at most 1 foot of elevation change at the top of this headscarp. Near the southern margin of the property, a west-east trending approximately 3-foot break in slope was noted, corresponding to what was initially interpreted to be the headscarp for the landslide mapped by Sorensen and Crittenden, Jr. (1979) and Western Geologic (2012). Irregular, slightly hummocky topography was noted to the south of this feature. Subsequent data collected from the test pit excavations would later show this to be an internal scarp within the larger landslide mass mapped by Sorensen and Crittenden, Jr. (1979). No other distinct geomorphic features indicating adverse geologic conditions were noted on or adjacent to the property during the site reconnaissance and geologic mapping.

Along the road cut for Horizon Run on the northeastern side of the property, dolomite bedrock was exposed. Similar rock was exposed on the northern road cut for North Powder Ridge Road along the northeastern margin of the property, and an outcrop of the dolomite bedrock is present immediately northeast of the bridge just east of the eastern intersection of Horizon Run with North Powder Ridge Road. The remainder of the property was found to be overlain by colluvial surficial materials, including sporadic boulders of red to purple quartzite derived from the underlying disaggregated Wasatch Formation. Surficial boulders were found to be as much as 2 feet in diameter.

5.2 GEOLOGIC HAZARD ASSESSMENT

The purpose of the geologic hazard assessment was to determine if any adverse geological structures were present on the property, and to assess the suitability of development of the Horizon Neighbourhood property from a geologic hazard standpoint.

Geologic hazard assessments are necessary to determine the potential risk associated with particular geologic hazards that are capable of adversely affecting a proposed development area. As such, they are essential in evaluating the suitability of an area for development and provide critical data in both the planning and design stages of a proposed development. The geologic hazard assessment discussion in the following paragraphs is based upon both qualitative and quantitative assessment of the risk associated with a particular geologic hazard, based upon the data reviewed and collected as part of this investigation.

A “low” hazard rating is an indication that the hazard is either absent, is present in such a remote possibility so as to pose limited or little risk, or is not anticipated to impact the project in a negative way. Areas with a low-risk determination for a particular geologic hazard generally do not require additional site-specific studies or associated mitigation practices with regard to the geologic hazard in question. A "moderate" hazard rating is an indication that the hazard has the capability of adversely affecting the project at least in part, and that the conditions necessary for the geologic hazard are present in a significant, though not abundant, manner. Areas with a moderate-risk determination for a particular geologic hazard may require additional site-specific studies and associated mitigation practices in the areas that have been identified as the most prone to susceptibility to the particular geologic hazard. A "high" hazard rating is an indication that the hazard is very capable of adversely affecting the project, that the geologic conditions pertaining to the particular hazard are present in abundance, and/or that there is geologic evidence of the hazard having occurred at the area in the historic or geologic past. Areas with a high-risk determination generally always require additional site-specific hazard investigations and associated mitigation practices. For areas with a high-risk geologic hazard, simple avoidance is often considered.

5.2.1 Landslide/Soil Creep

Soil creep and landslide hazards pose the most risk to development in the Horizon Neighbourhood property. Soil creep was found to be most prevalent on the steeper slopes of the western half of the property. Aspen trees in the northeastern part of the property were observed to exhibit some downslope basal bending of the trunks, and slickensided paleosols overlying bedrock in TP-1, TP-2, TP-3, and TP-9 provide additional evidence for this phenomenon. Though most of the excavations on the eastern and southern part of the property did not exhibit much soil creep evidence, the presence of generally shallow groundwater and fat clay paleosols or pond clay in other excavations on this part of the property can provide the means for soil creep occurrence in the future. Additionally, a kinked sand lens in TP-12 (Unit 5; see Figure A-13) may be indicative of soil creep occurring in the landslide area where the slope grade begins to significantly increase in the southernmost part of the property. Given this data, the risk associated with soil creep occurring on all parts of the property is considered to be high.

According to Sorensen and Crittenden, Jr. (1979), PH-2 and TP-11 were spotted on the northern margin of the Holocene landslide, and PH-3, TP-6, TP-7, TP-8, and TP-12 were spotted within the Holocene landslide deposit. However, landslide evidence was most explicitly expressed in TP-8 and TP-12.

TP-8 displayed shallow dolomite bedrock that was in direct contact on the upslope side with bedded, steeply dipping (75°E) paleosol/transitional units. The dolomite bedrock orientation in TP-8 (striking N70°W and dipping 22°NE) was consistent with the strike and dip found on bedrock outcrops across the Powder Mountain area, and specifically with the closest outcrop immediately northeast of the closest ski bridge (N58°W, 13°NE), located approximately 540 feet northeast of TP-8. As such, it was concluded that this bedrock has been stationary, and the transitional units have slid in reference to the bedrock. The absence of a weathering rind and associated slickensides around the bedrock indicates that the contact has not been produced by soil creep. Given the largely unaltered state of the landslide materials, it is likely that these materials were part of a slump block that terminated against the bedrock outcrop. The landslide that produced these observed features in TP-8 is considered to be Pleistocene-aged. This is due to the fact that there are no surficial geomorphic features to indicate a landslide in the subsurface, and a continuous colluvium unit and topsoil overlie the landslide materials in a manner consistent with the modern slope. As similar conditions were not encountered in any other excavation, this particular landslide/slump is considered to be limited in size and largely localized upslope of the TP-8 area.

TP-12 exhibited the most chaotic appearance of any of the excavations. Individual units were not consistent in thickness or character through the test pit, large bedrock clasts were found rafted within what was originally identified as an alluvial unit, voids were found below some of the bedrock clasts, and a sand lens was observed to have several kinks in it downslope. The test pit was spotted in some of the most irregular topography around the property, and combined with the subsurface data, it was confirmed that the test pit was spotted within a landslide. As dolomite bedrock was not encountered in this test pit, the landslide deposit is considered to extend to at least the depth of 14 feet. The landslide is considered to be Late Pleistocene to Holocene-aged, as it is uncertain whether the colluvium unit was part of the landslide, or superimposed upon a highly irregular landslide surface. The absence of dolomite bedrock clasts in the colluvium (though they are present in the underlying alluvium units) suggests the latter (Late Pleistocene age) for the age of the landslide, as dolomite clasts are likely to have been mixed into the colluvial unit, if the colluvial unit was part of (and therefore the same age as) the landslide. Because the features in TP-12 are unique to TP-12, the landslide may be largely localized to the TP-12 area.

Between TP-1 and TP-2, the Wasatch Formation is not present. From a cross-sectional standpoint, the material observed could be construed to be landslide deposits; however, evidence of landslide was not observed in either test pit. Therefore, the earth materials encountered in these two test pits are interpreted to be colluvial.

Though not as explicit as TP-8 or TP-12, subsurface features indicative of at least small-scale mass movement were also observed in TP-7 and TP-11. TP-7 was an anomalously weak test pit that exhibited continuous sloughing during logging, and also had individual units/subunits that dipped both consistent with (downslope side of test pit) and opposed to (upslope side of test pit) the modern slope (possible slump). TP-11 exhibited the similar possible slump feature of individual beds dipping with the modern slope on the downslope side of the pit, and dipping into the modern slope on the upslope side of the test pit. The colluvium and Wasatch Formation units were also seen to dip steeper than the modern slope at the end of the test pit on the downslope side.

Given that the subsurface data largely confirms the presence of a single large deposit or series of smaller landslide deposits on the eastern half of the property, and because groundwater can be found at shallow depths (at least in some parts of the year), the landslide hazard for this part of the property is considered to be moderate to high. Due to the fact that the hard dolomite bedrock is generally shallow, there is an absence of recent shearing, the modern slope is largely gentle, and the landslide deposit(s) are older and more subdued, appropriate mitigation practices may be able to reduce the landslide hazard risk associated with this part of the property to moderate or low.

The landslide hazard risk associated with the western half of the property is considered to be moderate, as evidence of recent shear (soil creep), the steepness of the slope, and shallow groundwater conditions provide conditions conducive to allowing the mass movement process to increase from a creep to a slide. Appropriate mitigation practices may be able to reduce the landslide hazard risk associated with this part of the property to low.

5.3 SEISMICITY

Following the criteria outlined in the 2012 International Building Code (IBC, 2012), spectral response at the site was evaluated for the *Maximum Considered Earthquake* (MCE) which equates to a probabilistic seismic event having a two percent probability of exceedance in 50 years (2PE50). Spectral accelerations were determined based on the location of the site using the *U.S. Seismic "DesignMaps" Web Application* (USGS, 2012); this software incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data developed for the United States by the U. S. Geological Survey as part of NEHRP/NSHMP (Frankel et al., 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2012).

Table 5.3
Short- and Long-Period Spectral Accelerations for MCE

Parameter	**Short Period (0.2 sec)**	**Long Period (1.0 sec)**
MCE Spectral Response Acceleration (g)	$S_S = 0.829$	$S_1 = 0.276$
MCE Spectral Response Acceleration Site Class B (g)	$S_{MS} = S_sF_a = 0.829$	$S_{M1} = S_1F_v = 0.276$
Design Spectral Response Acceleration (g)	$S_{DS} = S_{MS}*^2/_3 = 0.553$	$S_{D1} = S_{M1}*^2/_3 = 0.184$

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects of soft soils and is based on the average shear wave velocity of the upper 100 feet; based on our field exploration and our understanding of the geology in this area, the subject site is appropriately classified as Site Class B (*rock*). Based on IBC criteria, the short-period (F_a) and long-period (F_v) site coefficients are both 1.0. Based on the design spectral response accelerations for a *Building Risk Category* of I, II, III, or IV, the site's *Seismic Design Category* is D. The short- and long-period *Design Spectral Response Accelerations* are presented in Table 5.3; a summary of the *Design Maps* analysis is presented in Appendix D. The *peak ground acceleration* (PGA) may be taken as $0.4*S_{MS}$.

5.4 OTHER GEOLOGIC HAZARDS

Geologic hazards can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property. These hazards must be considered before development of the site. There are several hazards in addition to landslides, seismicity and faulting that, if present at the site, should be considered in the design of roads and critical facilities such as structures designed for human occupancy. IGES has assessed the potential for the presence of other geologic hazards, including liquefaction, rockfall, surface fault rupture, and debris flow and flooding; based on the observed geology, hydrology, stratigraphy, and topography, the potential for these geologic hazards impacting the site is considered low. Detailed discussions about these potential hazards are presented in the following paragraphs.

5.4.1 Liquefaction

The site is largely underlain by dolomite bedrock and disaggregated Wasatch Formation conglomerate. Given the generally very coarse and relatively thin nature of the surficial materials, and consistent with the existing geologic literature for the area, the risk associated with earthquake-induced liquefaction is expected to be low. However, both shallow groundwater and granular soils were observed to be present on the property; therefore, we cannot preclude the possibility for liquefaction to occur locally onsite. If liquefaction should occur at this site, it is expected to be a highly localized phenomenon.

5.4.2 Rockfall

IGES observed that there are no cliffs or exposed outcrops on steep slopes or other geomorphic features that would result in a rockfall hazard at the site. Therefore, the rockfall hazard for the property is considered to be low.

5.4.3 Surface Fault Rupture

There are no active or inactive faults currently mapped on, or trending toward the site (Sorensen and Crittenden, Jr. (1979); UGS and USGS (2006)). Therefore, the risk associated with surface fault rupture hazard for the property is considered to be low.

5.4.4 Debris Flow and Flooding

Debris flows and flooding typically occur on alluvial fans or in drainage channels that have been active in the Holocene and/or are currently active and associated with areas that include a drainage basin. The site is located near the top of the mountains that comprise the Powder Mountain Ski Resort. Major debris flow sources are absent and the site is not associated with a major drainage channel or a drainage basin. It is our judgment, therefore, that the potential for the site to be impacted by debris flows or flooding is considered low.

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations presented in this report are incorporated into the design and construction of the project. Evidence of soil creep has been observed throughout the site; our observations and analysis indicates the soil creep is occurring at the interface between overlying surficial soils (colluvium) and underlying bedrock units (Nounan Dolomite and Wasatch Formation conglomerate). The depth from existing grade to competent bedrock is generally on the order of 10 to 15 feet, although the depth to bedrock may be deeper, or shallower, locally. In consideration of the presence of creeping soils, and the presence of shallow surficial landslides, all on-grade structures must be supported on drilled piers anchored into bedrock. Conventional spread footings may be allowed in limited cases where a) the structure will be founded directly on either dolomite or conglomerate bedrock, and b) the foundation wall is designed to resist the passive resistance of the soil (to account for creep effects). However, we anticipate most, if not all structures will be founded on drilled piers. For areas where the depth of surficial soil overlying bedrock is greater than 20 feet, construction is not recommended.

The following sub-sections present our recommendations for general site grading, design of foundations, and moisture control.

6.2 EARTHWORK

Prior to the placement of foundations, general site grading is recommended to provide proper support for foundations, exterior concrete flatwork and concrete slabs-on-grade. Site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential settlement of foundations as a result of variations in subgrade conditions.

6.2.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris and undocumented fill (if any) should be removed. Any existing utilities should be re-routed or protected in-place. Tree roots may be encountered and should be grubbed-out and replaced with engineered fill if exposed in the foundation excavation. The foundation excavation should be assessed for soft or loose soils; any soft/loose areas should be compacted in place if the depth is less than 12 inches or removed and replaced with structural fill as recommended in this report.

6.2.2 Excavations

Soft, porous, or otherwise unsuitable soils beneath foundations or concrete flatwork may need to be over-excavated and replaced with structural fill. If over-excavation is required, the excavations should extend a minimum of 1 foot laterally for every foot of depth of over-excavation.

Excavations should extend laterally at least two feet beyond slabs-on-grade. Structural fill should consist of granular materials and should be placed and compacted in accordance with the recommendations presented in this report.

Prior to placing engineered fill, all excavation bottoms should be scarified to at least 6 inches, moisture-conditioned as necessary to at or slightly above optimum moisture content (OMC) and compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM D-1557 (modified Proctor). The scarification recommendation need not apply where competent bedrock is exposed.

6.2.3 Excavation Stability

The contractor is responsible for site safety, including all temporary slopes and trenches excavated at the site and design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by Occupational Safety and Health (OSHA) standards to evaluate soil conditions. Based on our observations, soil types may vary at this site but are expected to consist primarily of *Type B* soils (lean clay, fat clay). Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on OSHA guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions or groundwater is encountered, or when the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. Sloping of the sides at 1H:1V (45 degrees) in *Type B* soils may be used as an alternative to shoring or shielding. Excavating slopes at 1.5H:1V is recommended where coarse, granular soils are encountered (sand and gravel).

6.2.4 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements, should consist of structural fill. Structural fill may consist of excavated onsite soils and/or bedrock, or an approved imported granular soil. Within five feet of foundations or pavement the fines should have a liquid limit less than 25 and plasticity index less than 7. Structural fill should be free of vegetation and debris, and contain no rocks larger than 4 inches in nominal size (6 inches in greatest dimension). Soils not meeting the aforementioned criteria may be suitable for use as structural fill but must be approved by IGES prior to use. However, soil classifying as Fat CLAY (CH) (based on USCS classification) are generally not suitable for use as structural fill, with the exception that Fat CLAY may be used in roadway embankments provided it is placed at least 6 feet below pavement subgrade (bottom of aggregate section, measured vertically).

All structural fill should be placed in maximum 8-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 10-inch loose lifts if compacted by light-duty rollers, and maximum 12-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. These values are *maximums*; the Contractor should be aware that thinner lifts may be necessary to achieve the required compaction criteria. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill placed beneath footings and pavements should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. The moisture content should be at or slightly above the OMC for all structural fill – compacting dry of optimum is discouraged. Any imported fill materials should be approved by IGES prior to importing. Also, prior to placing any fill, the excavations should be observed by IGES to assess whether unsuitable materials have been removed. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

In addition, all utility trenches backfilled below pavement sections, curb and gutter and concrete flatwork, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches, including landscape areas, should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557).

Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

6.2.5 Oversized Material

If desired, oversize material (cobbles and boulders, at least 6 inches in greatest dimension) may be included in structural fill if they are placed in a manner that will not result in voids, loose soils, or uncompacted soils. These oversized particles should not be placed within 5 feet of the top of any embankment or within 5 feet of the outer slope of the embankment. If oversized particles are used in structural fill as discussed above, it is imperative that the contractor place and compact fill around oversized particles in accordance with the recommendations presented in the previous paragraphs. In addition to these recommendations, it is likely that the contractor will be required to use small compaction equipment such as hand operated jumping jack compactors to compact the structural fill within 2 feet of the oversized particle. We also recommend that a qualified geotechnical engineer or technician observe placement and compaction around oversized particles.

6.2.6 Erosion Control

Consideration should be given to the use of erosion control fabrics/waddles to facilitate the growth of vegetation on all cut and fill slopes. We recommend that the contractor give consideration to covering embankment fill, fill slopes, or cut slopes with topsoil that was removed during clearing and grubbing activities. The surface of the slope should be rough so that when the topsoil is placed, it will not be easily eroded and transported during snowmelt or wet seasons. The topsoil should be placed in a single 4-inch thick lift and track-walked with a dozer or hoe. Topsoil should be placed on slopes that are no steeper than 2H:1V. The track marks left by the dozer should not be flattened and should serve as areas to collect water and seeds to aid in growing native vegetation on the man-made slopes. An approved seed mix should be used in growing vegetation on man-made slopes, cuts, and other disturbed areas.

6.3 FOUNDATIONS

Evidence of soil creep has been observed throughout the site; our observations and analysis indicates the soil creep is occurring at the interface between overlying surficial soils (colluvium) and underlying bedrock units (Nounan Dolomite and Wasatch Formation conglomerate), and where landslide deposits have been identified. The depth from existing grade to competent bedrock is generally on the order of 10 to 15 feet, although the depth to bedrock may be deeper, or shallower, locally. In consideration of the presence of creeping soils, and the presence of shallow surficial landslides, all on-grade structures must be supported on drilled piers anchored into bedrock. Conventional spread footings may be allowed in limited cases where a) the structure will be founded directly on either dolomite or conglomerate bedrock, and b) the foundation wall is designed to resist the passive resistance of the soil (to account for creep effects). However, we anticipate most, if not all structures will be founded on drilled piers. IGES should review proposed structures on spread footings on a case-by-case basis to assess suitability of this foundation system for specific cases. For areas where the depth of surficial soil overlying bedrock is greater than 20 feet, construction is not recommended.

Within the areas designated as landslide deposits or areas where significant evidence of soil creep has been observed, it is possible to build a structure and maintain life safety if the structure is founded on drilled piers embedded into competent dolomite bedrock. However, future ground movement has the potential to damage roads, damage utilities, and could cause structures to become uninhabitable. The Owner should consider this risk before development of these areas. Plate A-3 indicates areas that have been delineated as either shallow landslide deposits or areas where significant soil creep has been identified (the area in pink). Within these areas, drilled piers must be embedded into competent dolomite bedrock. If a structure if mapped across the limits of the higher-risk area, the more conservative approach must be undertaken (e.g., the structure straddling the limit line must be founded into dolomite). Areas outside of the pink area must also be founded on drilled piers; however, the drilled piers may be embedded either in dolomite bedrock or a minimum of 10 feet into competent Wasatch Formation conglomerate.

For purposes of construction, identification of dolomite bedrock should be straight-forward; the dolomite is generally very hard, bluish-gray, and homogenous. It should be noted that the uppermost 5 to 10 feet of the dolomite bedrock is typically highly weathered, and may have intervening thin (< 1-foot-thick) clay beds. It is for this reason that it is recommended that the drilled piers be embedded into the competent dolomite bedrock, which may be as much as 10 feet below the top of where weathered bedrock is first encountered. Differentiating between Wasatch Formation bedrock and the overlying colluvial soils (derived from the Wasatch Formation bedrock), however, may be difficult; therefore, where drilled piers will be founded into Wasatch Formation, a representative from IGES should observe the drilled hole prior to placement of steel or concrete to assess whether the minimum embedment into Wasatch Formation has been achieved.

The following paragraphs summarize our recommendations for conventional spread footings and deep foundations.

6.3.1 Spread Footings

In limited cases where a structure can be founded *entirely* on bedrock (either dolomite or conglomerate), spread footings may be utilized upon written approval of IGES for specific cases. Bedrock/soil or fill/native transition zones are not allowed. If differing earth materials are exposed in the footing excavations, then the footings should be deepened such that all footings bear on the same earth materials (e.g., all footings bear on the same type of bedrock). Alternatively, the building pad may be over-excavated a minimum of 2 feet below the bottom of proposed footings and replaced with structural fill, such that the footings bear entirely on a uniform fill blanket (note that the bottom of footing must still be below the depth of observed surficial soils). Where utilized, all fill beneath the foundations should consist of structural fill and should be placed and compacted in accordance with our recommendations presented in Section 6.2.4 of this report.

In conjunction with the use of spread footings, the foundation wall on the uphill-side of the structure must be designed to resist passive earth pressures. For this case only, the passive earth pressure must be provided by IGES on a case-by-case basis. Alternatively, a pressure relief wall can be constructed to eliminate lateral earth pressures from the foundation wall (typically would consist of a soldier pile wall or a soil nail wall).

Shallow spread or continuous wall footings constructed on competent bedrock (dolomite or conglomerate) may be proportioned utilizing a maximum net allowable bearing pressure of **4,500 pounds per square foot (psf)**. However, if the foundations are underlain by a minimum of 2 feet of structural fill or competent native soils, a maximum net allowable bearing pressure of **2,600 psf** should be used for design. The net allowable bearing values presented above are for dead load plus live load conditions. The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings. The allowable bearing capacity may be increased by one-third for short-term loading (wind and seismic). Higher bearing capacities may be allowed where the entire structure is founded on dolomite bedrock; however, considering the depth at which dolomite was observed, this scenario appears unlikely.

All foundations exposed to the full effects of frost should be established at a minimum depth of 42 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost (e.g., *a continuously heated structure*), may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes.

6.3.2 Drilled Piers

Where habitable structures will be founded over soils exhibiting evidence of landslide or creep, structures shall be constructed on drilled piers. The drilled piers shall extend through the upper surficial soils (soils subject to creep) and embed within competent dolomite bedrock or Wasatch Formation conglomerate bedrock. However, it is anticipated that in most cases, embedment into dolomite will be the preferred practice.

The purpose of the drilled piers is to resist lateral forces that arise from soil creep, which effectively represent full passive soil resistance within the creep zone. This lateral force includes both the load of the soil directly on the drilled pier (a distributed load), plus the load transferred from the grade beams to the drilled piers (effectively a point load). For our design, we have assumed an allowable lateral deflection of 2 inches. Drilled pier reinforcement shall be designed by the structural engineer. The structural engineer must also evaluate the designs presented herein to verify the drilled piers are structurally sound with respect to shear loads and moments (e.g., confirm that the drilled piers will not break under design lateral loads). For this design, IGES has considered the following criteria:

- Allowable lateral deflection: 2 inches
- Unit weight of soil: 120 pcf
- Ground surface slope: 20 degrees
- Passive lateral earth pressure coefficient: Kp = 8.355
- Grade Beams – 2.5 feet depth, 1-foot below grade, maximum spacing between piers is 24 feet, load from soil creep is 5,640 lb/LF
- Maximum distance between piers is 24 feet

The design of the drilled piers will vary, depending on the depth of potentially creeping soils or landslide deposits (e.g., the depth from finish grade to bedrock). IGES recommends the following guidelines for design of drilled piers.

Table 6.3.2-1
Recommended Drilled Pier Lengths and Diameters – Outside Piers

Depth to Bedrock (ft)	Drilled Pier Diameter (ft)	*Min. Embedment into Dolomite (ft)	Total Pier Length (ft) (finish grade to toe)
≤ 12	32	6	≤ 18
14	32	8	22
16	36	8	24
18	42	8	26
20	42	10	30
>20	Construction Not Recommended		

Table 6.3.2-2
Recommended Drilled Pier Lengths and Diameters – Inside Piers

Depth to Bedrock (ft)	Drilled Pier Diameter (ft)	*Min. Embedment into Dolomite (ft)	Total Pier Length (ft) (finish grade to toe)
≤ 12	32	6	≤ 18
14	36	8	22
16	42	8	24
18	42	8	26
20	48	10	30
>20	Construction Not Recommended		

*Where Wasatch Formation conglomerate is present, minimum embedment is 10 feet for all cases. If structure is in the 'Pink' zone delineated on Plate A-3, the drilled piers must be embedded into dolomite.

Table 6.3.2-3
Allowable Axial Capacity of Drilled Piers* - Dolomite

Depth to Bedrock (ft)	Drilled Pier Diameter (ft)	Allowable Capacity Compression (kips)	Allowable Capacity Tension (kips)
≤ 12	32	45,000	27,000
14	32	55,000	39,000
16	36	65,000	44,000
18	42	82,000	52,000
20	42	94,000	67,000

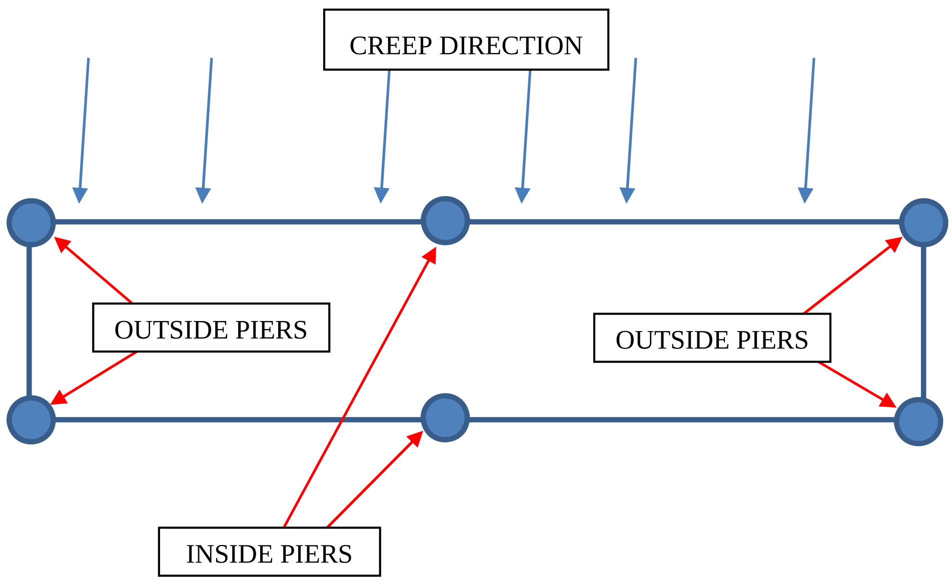
*Geotechnical capacity reported, verify structural capacity with structural engineer

Table 6.3.2-4
Allowable Axial Capacity of Drilled Piers* - Conglomerate

Drilled Pier Diameter (ft)	Allowable Capacity Compression (kips)	Allowable Capacity Tension (kips)
32	1,058	722
36	1,257	815
42	1,584	954
48	1,944	1,096

*Geotechnical capacity reported, verify structural capacity with structural engineer. Assumed min. 10 feet embedment into conglomerate.

Table 6.3.2-1 is for outside piers, and Table 6.3.2-2 is for inside piers:



Sample design calculations are presented in Appendix E. Pier reinforcement should be designed by the structural engineer.

6.4 SETTLEMENT

6.4.1 Static Settlement

Static settlement of properly designed and constructed conventional foundations, founded as described above, are anticipated to be on the order of 1 inch or less. Differential settlement is expected to be half of the total settlement over a distance of 30 feet.

6.4.2 Dynamic Settlement

Based on the field data collected for this site, it is our opinion that the onsite native bedrock and/or clayey colluvium will exhibit negligible seismically-induced settlement during a MCE seismic event. Similarly, properly compacted structural fill is expected to exhibit negligible seismically induced settlement during a MCE seismic event.

6.5 SLOPE GRADING RECOMMENDATIONS

The following generalized recommendations are for engineered slopes (cut slopes and fill slopes). Recommendations for grading of engineered slopes are intended to minimize the potential for future surficial failures. For purposes of this report, surficial failure includes excessive erosion, sloughing, slumping, mass wasting, rockfall, and similar relatively shallow failures.

We recommend fill slopes taller than 10 feet be constructed as a buttress fill, as illustrated on Figure F-1. These recommendations are expected to pertain largely to the area around the lodge, but may apply anywhere a fill slope taller than 10 feet will be constructed. General recommendations for construction of buttress fills are presented in the following sections:

6.5.1 General Specifications

Cut and fill slopes should be constructed no steeper than 2H:1V. All cut slopes should be assessed geologically by IGES during grading to verify the geologic conditions upon which the following recommendations were made. It is feasible that cut and fill slopes may be constructed at slopes steeper than 2H:1V provided the slope is structurally stabilized; stabilization measures may include products such as an *Anchor Reinforced Vegetated System* (ARVS) (e.g., Xtreme Armor System by Western Excelsior), gabions, anchored shotcrete, or another similar system. If slopes steeper than 2H:1V are desired, IGES should be consulted to provide slope-specific recommendations and design guidelines.

Buttress fills should be constructed with a keyway (see Figure F-1). In general, the keyway back cut should be constructed no steeper than 1.5H:1V gradient, assuming the back cut will have a minimum factor of safety of 1.2. Flatter back cuts will reduce the potential for back cut failures. In order to decrease the risk of back cut failure, cut slopes should be off-loaded prior to excavating the buttress back cut. In addition, the amount of time the back cut remains exposed and unsupported should be minimized to reduce the risk of back cut failure. All stability fills should be a minimum of 10 feet wide (equipment width) at the top of the slope.

6.5.2 Keyway Sizing

As a minimum, keyways should be excavated 2 feet below toe grade; deeper keyway excavations may be necessary, depending on the height of the slope and prevailing geologic conditions. *The minimum keyway depth for the fill slope associated with the lodge structure is 5 feet, minimum* *length is 25 feet*. The width of a keyway is measured horizontally from the toe of slope (top of front cut) to the toe of the back cut (heel), with a 2 percent drop to the heel. The depth of a keyway is measured from the toe of the fill slope to the bottom of the keyway. The minimum width of a keyway is 8 feet, except as allowed by IGES for specific cases; wider keyways may be needed if geologic conditions warrant (as noted above, a 25-foot keyway is required for the lodge structure). Adjustments to keyway width may be allowed if shallow bedrock is encountered; IGES should approve any adjustments and should evaluate bedrock/grading conflicts on a case-by-case basis.

6.5.3 Drainage

All excavations for fill slopes taller than 15 feet should be provided with a subdrain at the heel to reduce the potential for infiltrating water to perch and migrate toward the slope face; a typical heel subdrain is detailed on Figure F-1. Subdrains placed along the back cut of fill slopes may be constructed with 3-inch perforated PVC pipe, surrounded by approximately 6 cubic feet per lineal foot of ¾ inch gravel, wrapped in permeable filter material. Subdrains should be provided with outlet drains every 100 feet; for a slope less than 100 feet in length, an outlet at either end of the slope is recommended. All subdrains should be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys. Some modification to the drainage recommendations presented herein may be feasible; however, any change should be approved by IGES prior to implementation.

6.5.4 Benching

Where fills are to be placed on ground with slopes steeper than 5H:1V, the ground shall be stepped or benched (see Figure F-2 for a graphic illustration). At a minimum, benches should be constructed every four (4) vertical feet. Benches shall be excavated a minimum lateral depth of four (4) feet into competent material or as otherwise recommended by IGES. However, the *lowest* bench should be excavated a minimum lateral depth of 8 feet into competent material (effectively creating a keyway).

6.5.5 Slope Protection

Slope planting and other measures should be provided immediately following construction. Slope protection polymers, straw waddles, and/or jute mesh should also be considered to limit the amount of erosion on slopes subject to erosion until landscaping and other permanent erosion protection measures are fully in place.

6.5.6 Earthwork Recommendations

In addition to the normal compaction procedures for structural fill specified in Section 6.2.4, compaction of fill slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to IGES. As an alternative to slope compaction, slopes may be constructed 2 to 3 feet ‘fat’ and trimmed back using a bulldozer with a slope board or similar equipment. Upon completion of grading, relative compaction of the fill out to the slope face shall be at least 90 percent of the maximum dry density per ASTM D 1557 (modified Proctor).

6.5.7 Rockeries

For rockeries with a single tier up to 8 feet in height, or a two-tier rockery where neither tier is taller than 8 feet and having a relatively flat back slope, the Contractor may follow the *Rockery Construction Guidelines* letter prepared by IGES (2013). For taller rockeries, or rockeries having more than two tiers, project-specific design will be required.

6.6 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.35 for undisturbed earth materials or structural fill should be used. A higher coefficient of friction may be used for specific locations where coarse/granular soils or bedrock have been documented at the foundation grade; structure-specific recommendations by IGES should be made prior to using a higher value.

Ultimate lateral earth pressures from natural soils and *granular* backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 6.6.

Table 6.6
Recommended Lateral Earth Pressure Coefficients

Condition	Level Backfill		2:1 Backfill	
	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active (Ka)	0.26	31.2	0.37	44.4
At-rest (Ko)	0.41	49.5	0.58	70
Passive (Kp)	3.85	462	-	-

The coefficients and densities presented in Table 6.6 assume no buildup of hydrostatic pressures. The force of the water should be added to the presented values if hydrostatic pressures are anticipated.

Clayey soils drain poorly and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures. Therefore, clayey soils should not be used as retaining wall backfill. Backfill should consist of either native granular soil or sandy imported material with an Expansion Index (EI) less than 25.

Walls and structures allowed to rotate slightly should use the active condition; if the element is constrained against rotation (i.e., a basement wall) the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

6.7 CONCRETE SLAB-ON-GRADE CONSTRUCTION

To minimize settlement and cracking of slabs, and to aid in drainage beneath the concrete floor slabs, all concrete slabs should be founded on a minimum 4-inch layer of compacted gravel overlying structural fill or competent native earth materials. The gravel should consist of free draining gravel or road base with a 3/4-inch maximum particle size and no more than 5 percent passing the No. 200 mesh sieve. The layer should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. Where fat clay is observed (LL>50) we recommend a minimum over-excavation of 12 inches below subgrade (12 inches below the 4 inches of compacted gravel). The over-excavated fat clay should be replaced by granular structural fill.

Slab-on-grades may be designed using an allowable bearing stress of **500 psf** (dead plus live load) and a Modulus of Subgrade Reaction of **200 psi/inch**. It should be noted that the Modulus of Subgrade Reaction is not a function of soil properties alone but is also influenced by other factors, including the width of the loaded area, the shape of the loaded area, and the specific location under the slab. As such, the structural engineer should exercise care and engineering judgment when using the above stated value for design.

All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with a welded wire fabric, re-bar, or fibermesh. Slab reinforcement should be designed by the structural engineer. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. If slump and/or air content are beyond the recommendations as specified in the plans and specifications, the concrete may not perform as desired. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI).

A capillary break consisting of clean gravel or a moisture barrier (vapor retarder) consisting of 10-mil thick Visqueen (or equivalent) plastic sheeting should be placed below slabs-on-grade where moisture-sensitive floor coverings or equipment is planned. Prior to placing this moisture barrier, any objects that could puncture it, such as protruding gravel or rocks, should be removed from the building pad. Alternatively, the subgrade should be covered with 2 inches of clean sand.

6.8 MOISTURE PROTECTION AND SURFACE DRAINAGE

During Construction: Over-wetting the soils prior to, during, or after construction may result in softening and pumping, causing equipment mobility problems and difficulty in achieving compaction. Every effort should be taken to ensure positive drainage away from roadway areas to reduce the potential for water to migrate below pavements and concrete flatwork. The recommended minimum slope is two percent (2%) in pavement areas. Moisture should not be allowed to infiltrate the soils in the vicinity of, or upslope from, the roadways.

Slope Protection: To aid in maintaining surficial slope stability, we recommend that a water interceptor swale be constructed at the top of all engineered slopes (cut slopes, fill slopes). This swale should be designed to intercept all uphill slope drainage and divert the drainage around the slopes. The drainage should be controlled as it travels around the slopes and should be tied into the curb and gutter or other drainage system associated with the road.

Residential Structures: Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the home should be implemented. Structures that are located near the toe of ascending slopes may be subject to sheet flow during periods of heavy rain or snow melt. Therefore, the Civil Engineer may also wish to consider construction of additional surface drainage to intercept surface runoff, or a curtain drain to intercept seasonal groundwater flow, if any.

We recommend that desert or Xeriscape landscaping be considered within 5 feet of foundations. We further recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures or beyond the limits of backfill (whichever distance is greater). Irrigation valves should be placed a minimum of 5 feet from foundations and should always be placed beyond the limits of foundation backfill. The builder should be responsible for compacting the exterior backfill soils around the foundation in lifts no greater than 12 inches to 90 percent of the maximum dry density (ASTM D1557). Additionally, the ground surface within 10 feet of structures should be constructed so as to slope a minimum of five percent away. Pavement sections should be constructed to divert surface water off of the pavement into storm drains. Parking strips and roadway shoulder areas should be constructed to prevent infiltration of water into the areas surrounding pavement.

Foundation Drains: IGES recommends a perimeter foundation drain be constructed for any proposed structure with a subterranean component (e.g., a basement); the perimeter drain should be designed in accordance with guidelines presented in the International Residential Code (IRC).

6.9 SOIL CORROSION POTENTIAL

Laboratory test results from samples obtained from nearby test pits from our original project-wide geotechnical investigation indicate that near-surface native soils had a sulfate content ranging from

34 to 86 ppm (IGES, 2012b, TP-17 and TP-18). Based on soil conditions encountered during our field investigation and results of chemical testing, the soils are classified as having a 'low' potential for deterioration of concrete due to the presence of soluble sulfate. We recommend that conventional Type I/II Portland cement be used for all concrete in contact with site soils.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil, we have reviewed laboratory tests conducted for nearby soil samples obtained during our previous project-wide geotechnical investigation (IGES, 2012b, TP-17, TP-18). Two samples were tested for soil resistivity (AASHTO T288), soluble chloride content, and pH. The tests indicated that the onsite soil tested had a minimum soil resistivity of ranging from 980 to 2,200 OHM-cm, soluble chloride content ranging from 11 to 12 ppm, and a pH ranging from 6.3 to 6.5. Based on this result, the onsite native soil is considered *severely* corrosive to ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal that will be in contact with native clay soils.

6.10 PAVEMENT DESIGN

Near-surface soils encountered at the site consist largely of clayey soils, and are therefore expected to provide poor pavement support. Interior roadways for the project are expected to experience minimal traffic, primarily for access to the lodge and access to parking areas. Based on our assessment of the subgrade soils, the following pavement sections are presented to provide a 20-year design life for the subject roads. It should be noted that construction traffic will account for the majority of the loading during the life of the road.

Table 6.10
Pavement Recommendations

Type of Street	Asphalt (in.)	Roadbase (in.)	Subbase (in.)
Light Traffic (e.g. access to a carport)	3	6	8
Lodge Access Road	4	6	8

Earth materials classifying as Fat CLAY (CH) were identified onsite. Where fat clay is identified on the pavement subgrade, IGES recommends over-excavating an additional 12 inches and replacing with relatively frost-free granular materials (subbase or a pit-run gravel will generally fulfill this requirement). Because of the potential for Fat CLAY to exist beneath the roadway, it is imperative that the pavement section be constructed as recommended and that the pavement be designed to divert surface runoff to gutters and storm drains to minimize the risk of pavement distress arising from expansive soils and/or frost heave. The pavement should be constructed to divert water away from the center of the roadway with a minimum 2 percent slope towards the gutter. Our recommendation to overexcavate and remove the uppermost 12 inches of the Fat CLAY assumes that these moisture and drainage recommendations will be implemented. If these recommendations are not implemented or if poor asphalt quality allows the subgrade to become saturated, differential heave may occur which could cause distress to the pavement section.

Asphalt has been assumed to be a high stability plant mix and base course material composed of crushed stone with a minimum CBR of 70, and subbase (granular borrow) should have a minimum CBR of 30. Road base and subbase should be compacted to 95% of MDD as determined by ASTM D-1557 (Modified Proctor). Asphalt should be compacted to a minimum of 96 percent of the Marshall maximum density. Asphalt and aggregate base material should conform to local requirements. Subgrade should be scarified to a depth of 8 inches and compacted to 95% of MDD as determined by ASTM D-1557. Positive drainage away from roadways must be provided to minimize the potential for saturation of subgrade soils beneath constructed pavements.

Where Portland Cement Concrete (PCC) pavements are planned, such as near trash enclosures or other areas expected to support heavy truck traffic, we recommend a minimum of 6 inches PCC underlain by a minimum 6 inches of aggregate base course.

If conditions vary significantly from our stated assumptions (including stated traffic assumptions) IGES should be contacted so we can modify our pavement design parameters accordingly.

6.11 GEOLOGIC HAZARD ASSESSMENT – CONCLUSIONS & RECOMMENDATIONS

Based upon the geologic reconnaissance of the project area and the geologic conditions observed in the exploration test pits, geologic features in the form of shallow landslides, soil creep, and slumping have the potential to adversely impact the proposed development. Given the geologic evidence discussed herein, and the slope stability assessment addressed in Section 4.3, the following conclusions are made:

1. Evidence of soil creep was observed across the property; soil creep represents a high hazard for the site as a whole. Though a very slow-moving process, soil creep effects may require the annual inspection and/or maintenance of utilities. Soil creep is a relatively shallow phenomenon; structures can be founded over areas subject to soil creep provided the structure is founded on properly designed drilled piers embedded into competent bedrock.

2. Subsurface data collected from the test pits confirm the presence of the large landslide deposit initially mapped by Sorensen and Crittenden, Jr. (1979) and subsequently mapped by Western Geologic (2012) on the southeastern part of the property. The subsurface data suggest that the large deposit may be the product of a series of smaller slides or slumps that have a generally localized affected area. Additionally, a small headscarp was noted on the northern part of the

property. Though some of the data suggest an older (Pleistocene) age for the large landslide, the presence of shallow groundwater, generally poorly consolidated soils, and steeper slopes in the southern part of the property may provide conditions conducive to the rejuvenation of the slide. As such, the landslide hazard is considered to be moderate for the western half of the property and moderate to high for the eastern half of the property. Therefore, appropriate mitigation practices are required to be employed in order to make the site suitable for development from a landslide hazards standpoint.

3. Surface fault rupture, rockfall, debris flow, and flooding hazards are considered to be low for the property.

4. Published literature indicate that the liquefaction potential for the site is expected to be low. However, due to the presence of granular soils and shallow groundwater and the unknown character of the soils underlying those examined in the geotechnical report, the potential for liquefaction occurring at the site cannot be ruled out. If liquefaction should occur, the impact would be expected to be highly localized.

Given the conclusions listed above, IGES makes the following recommendations:

1. Based on the data collected from the test pit and pothole excavations, except with limited exceptions, all habitable structures should be founded on drilled piers anchored into bedrock (dolomite or conglomerate). All foundations within the 'pink' area as designated on Plate A-3 must be set into hard dolomite bedrock. Where surficial soils are greater than 20 feet in depth (measured from finish grade to the top of bedrock), construction of habitable structures is not recommended.

2. IGES should be present onsite during the foundation excavation and/or drilling to assess whether the foundations are emplaced into the appropriate material and to the appropriate depth.

3. Because the dolomite bedrock was not encountered at uniform depths across the property, and in some cases not encountered at all, the Owner may wish to consider that prior to the commencement of foundation excavation/drilling operations a drilling program be conducted across the property to further delineate the depth to bedrock. This will serve to refine the *Footing Delineation Map* (Plate A-3) and more clearly identify areas where construction is feasible and provide additional guidance with respect to drilled pier sizing for specific structures.

4. The observation of shallow groundwater on localized parts of the property makes necessary mitigation practices to adequately address this potential hazard. Appropriate grading

measures in low-lying areas susceptible to near-surface groundwater conditions is recommended. Temporary dewatering or the construction of land drains may be desired to help facilitate construction.

6.12 CONSTRUCTION CONSIDERATIONS

The following items of note should be brought to the attention of the Owner, and the Contractor who will be performing earthwork and/or building the foundations within the project area:

1. Habitable structures may be constructed on drilled piers such that life safety can be reasonably preserved. However, within areas susceptible to creep, and in particular areas mapped as landslide deposits, future ground movement may damage utilities, pavement, and other improvements. The Owner should understand and accept that some roadway and/or utility maintenance may be necessary if soil creep occurs, or if an existing landslide mass becomes reactivated. Use of flexible utilities that can accommodate some ground movement may serve to reduce the risk associated with future ground movement damaging utilities.

2. *For all foundations*, prior to placement of steel, concrete, or structural fill, IGES should assess the subgrade for the presence of adverse conditions, which may include (but not necessarily be limited to): a) transitions zones, b) soft/loose soil, or c) potentially adverse geologic structures. If identified, potentially adverse geologic structures will be brought to the attention of the Client for further review and input. In addition, IGES should evaluate whether the depth of embedment of drilled piers into bedrock is adequate.

3. Where the depth to bedrock is greater than 20 feet, construction using the methods discussed herein is not recommended. Where depth to bedrock is greater than 20 feet, construction may be feasible; however, design of ground improvement or other methods to stabilize the building envelope should be developed on a case-by-case basis and approved by IGES prior to implementation. Techniques such as the use of secant walls, or tangent walls, or mass-grading, could potentially be used to stabilize the subgrade where the depth to bedrock is greater than 20 feet.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations presented in this report are based on our limited field exploration, laboratory testing and understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is likely that variations in the soil and groundwater conditions exist between and beyond the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, we should be immediately notified so that we may make any necessary revisions to recommendations presented in this report. In addition, if the scope of the proposed construction changes from that described in this report, IGES should be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No warranty, expressed or implied, is made.

It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

7.2 ADDITIONAL SERVICES

The recommendations presented in this report are based on the assumption that an adequate program of tests and observations will be made during construction. IGES staff should be on site to assess compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Observation of foundation soils to assess their suitability for footing placement.
- Observation of soft/loose soils overexcavation.
- Observation of temporary excavations and shoring.
- Consultation as may be required during construction.
- Quality control and observation of concrete placement.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 748-4044.

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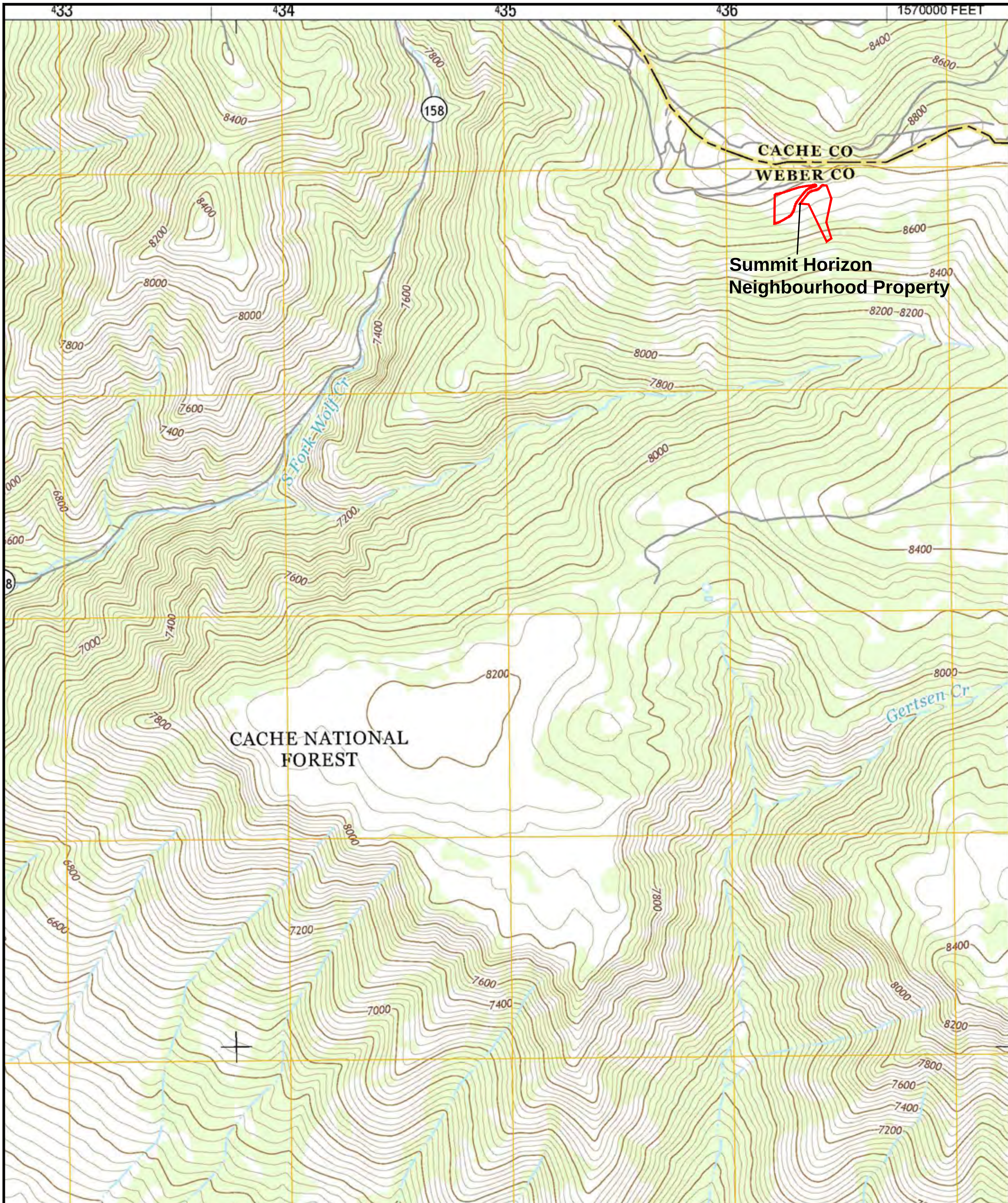
APPENDIX A



BASE MAP:

USGS Huntsville 7.5-Minute Topographic Quadrangle Map (2014)

QUADRANGLE LOCATION

N

0 1000 2000 3000

FEET

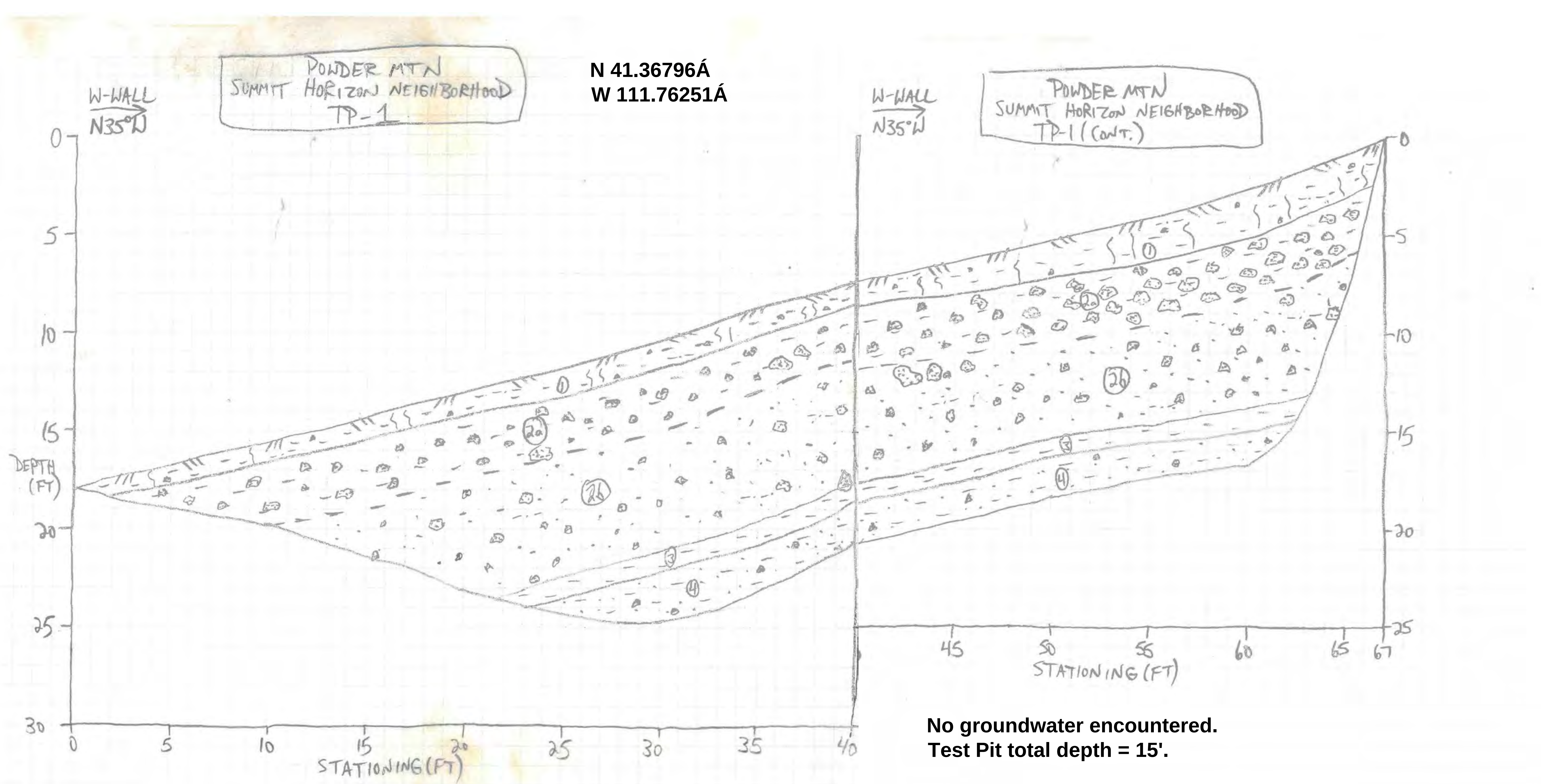
FIGURE A-1

SITE VICINITY MAP

SUMMIT HORIZON NEIGHBOURHOOD
GEOLOGIC AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/22/2016	SCALE:
PROJECT:01628–013	1"=2,000'

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1.5-2' thick; dark yellowish brown (10YR 4/2); lean CLAY, medium stiff, slightly moist, low plasticity, massive; some silt and occasional to common quartzite clasts up to 3" thick, though some boulders several feet in diameter noted on ground surface; clasts are subrounded to rounded, all white to pink quartzite; gravel and larger sized clasts comprise ~5% of unit; abundant plant and tree roots, though much more concentrated in uppermost 6" of unit.

2. 2A Colluvium 1 (Qcl): ~2-3' thick; dark yellowish brown (10YR 4/2) to moderate reddish brown (10R 4/6); lean CLAY with gravel, medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~50% of unit, all subrounded to subangular white to pink quartzite up to 1.5' diameter, though mode average 6-8"; abundant plant and tree roots; poorly sorted.

2. 2B Colluvium 2 (Qcl): ~5-6' thick; moderate red (5R 5/4) to dark yellowish orange (10YR 6/6) to moderate reddish brown (10R 4/6); poorly graded SAND with gravel, medium dense, moist, massive; gravel and larger sized clasts comprise ~40% of unit, all subrounded to subangular quartzite up to 1.5' diameter, though mode average 4-6"; abundant pinhole voids throughout; uppermost ~2' of subunit has abundant plant and tree roots, while basal ~3' of subunit has occasional plant and tree roots; poorly sorted; irregular, sharp basal contact.

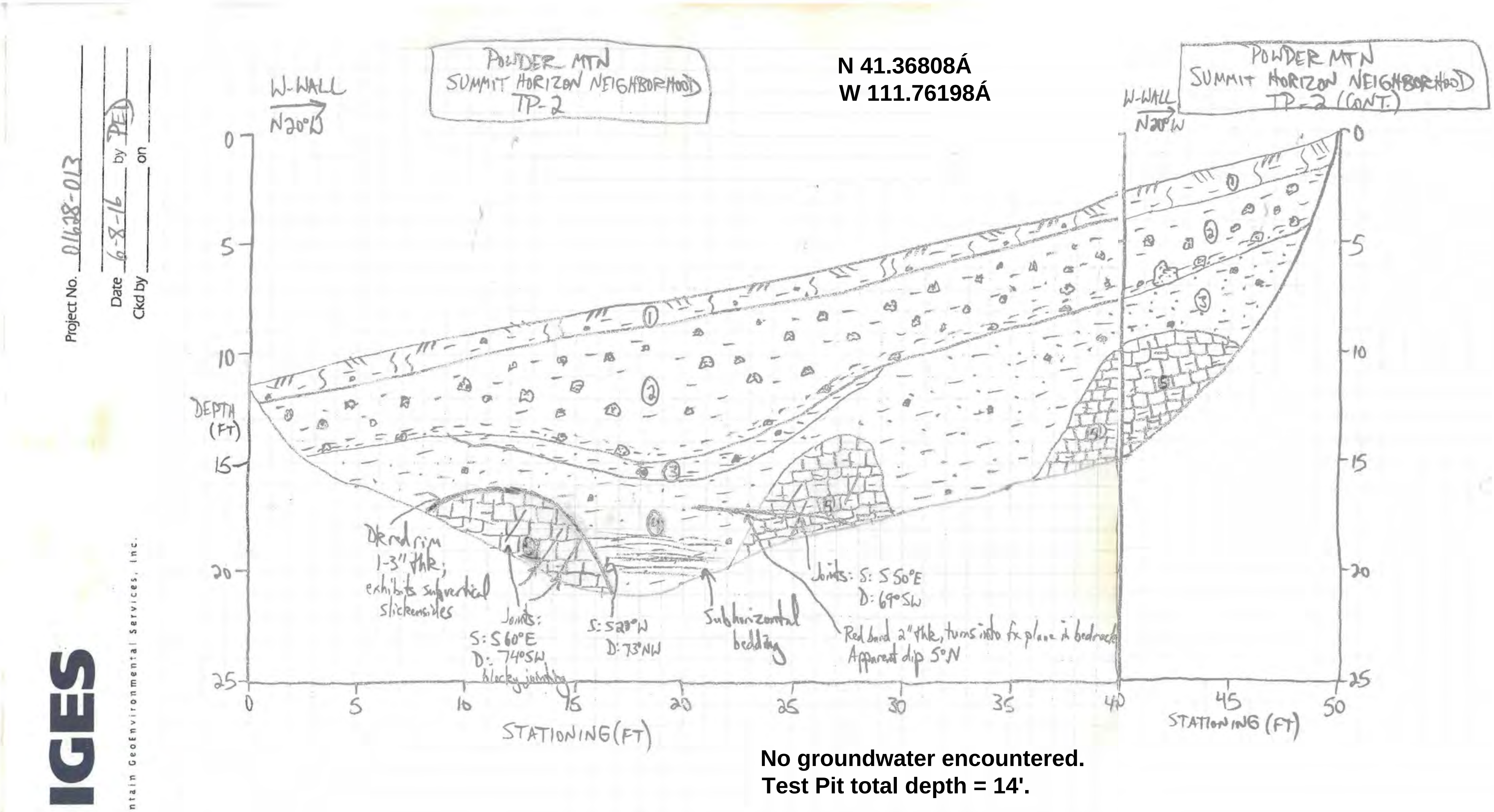
3. Paleosol?: ~1' thick; moderate red (5R 5/4) to brownish black (5YR 2/1); fat CLAY, medium stiff, moist, high plasticity, massive; abundant slickensides throughout; rare small (~1" diameter) dark gray weathered dolomite clasts; basal 1/2 of unit generally darker colored; parallels current topography; sharp, irregular basal contact; *top of unit likely shear plane for creep.

4. Weathered Bedrock (Cn): At least 2' thick; moderate reddish brown (10R 4/6) to dark gray (N3); sandy CLAY with common angular weathered dolomite bedrock clasts; gradational to clayey SAND; medium stiff, moist, moderate to high plasticity, massive; dark gray finely sparry dolomite bedrock clasts all angular, mottled, highly weathered, and up to 3" in diameter.

FIGURE A-2
TP-1 LOG

SUMMIT HORIZON NEIGHBORHOOD
GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016
FILE: 01628–013
SCALE: 1"=5'
IGES



No groundwater encountered.
Test Pit total depth = 14'.

LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1-1.5' thick; dark yellowish brown (10YR 4/2); lean CLAY, medium stiff, slightly moist, low plasticity, massive; some silt and occasional to common quartzite clasts up to 3" thick, though some boulders several feet in diameter noted on ground surface; clasts are subrounded to rounded, all white to pink quartzite; gravel and larger sized clasts comprise ~5% of unit; abundant plant and tree roots, though much more concentrated in uppermost 6" of unit.

2. 2A Colluvium 1 (Qcl): ~4.5' thick; dark yellowish brown (10YR 4/2) to moderate reddish brown (10R 4/6); lean CLAY with gravel, medium stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~50% of unit, all subrounded to subangular white to pink quartzite up to 1.5' diameter, though mode average 6-8"; abundant plant and tree roots; poorly sorted.

2. 2B Colluvium 2 (Qcl): ~6" thick; dark yellowish orange (10YR 6/6) to moderate yellowish brown (10YR 5/4);SILT with gravel, medium stiff, moist, massive; gravel and larger sized clasts comprise ~40% of unit, all subangular quartzite up to 6" in diameter, though mode average 2"; abundant pinhole voids throughout; poorly sorted; irregular, sharp basal contact.

3. Paleosol?: ~6"-1' thick; moderate red (5R 5/4) to brownish black (5YR 2/1); fat CLAY, medium stiff, moist, high plasticity, massive; generally the same as seen in TP-1 except largely devoid of clasts and fewer slickensides.

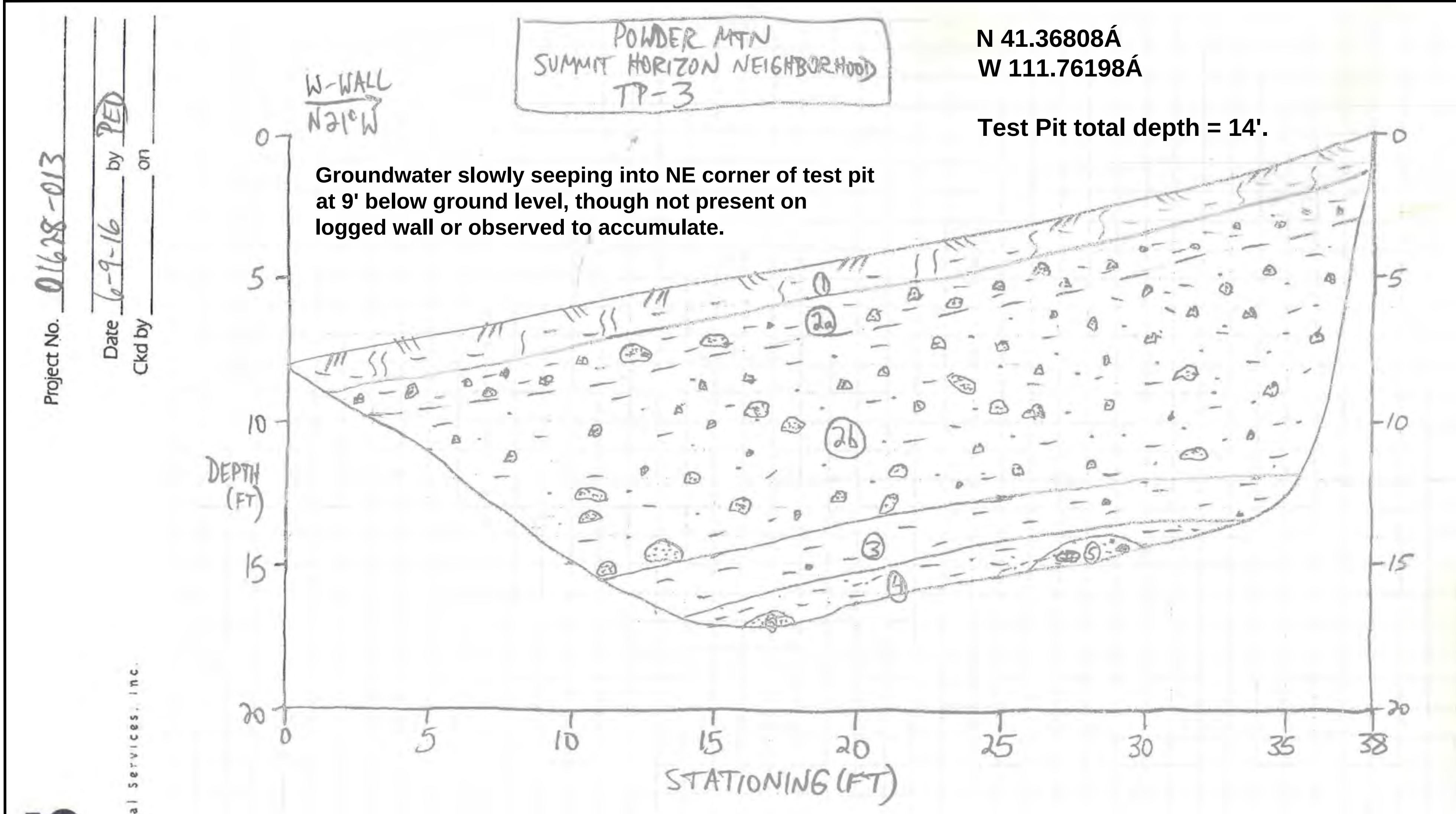
4. Highly Weathered Bedrock (Cn): ~2-5'+ thick; dark gray (N3) to moderate red (5R 5/4); mottled appearance; sandy CLAY, stiff, moist, moderate plasticity, thinly bedded; internal structure of original dolomite bedrock still evident; highly sandy where less weathered; occasional angular bedrock clasts up to 1" diameter.

5. Partially Weathered Bedrock (Cn): At least 5'+ thick; dark gray (N3) to medium dark gray (N4); mottled appearance; dolomite bedrock is soft to medium hard, and is sandy where more weathered; exhibits well-developed blocky jointing and largely subhorizontal thin bedding; contains slickensided red ring around southernmost bedrock knob.

FIGURE A-3
TP-2 LOG

SUMMIT HORIZON NEIGHBORHOOD
GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE: 1"=5'	IGES
FILE: 01628-013		



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1' thick; dark yellowish brown (10YR 4/2) to grayish brown (5Y 3/2); lean CLAY, medium stiff, slightly moist, low plasticity, massive; abundant plant and tree roots; occasional gravel and larger sized medium gray (N5) quartzite clasts up to 8" diameter.

2.2A Colluvium (Qcl): ~1.5-2' thick; dark yellowish brown (10YR 4/2); lean CLAY with gravel gradational to clayey GRAVEL, medium stiff to loose, slightly moist, low plasticity, massive; minor silt; gravel and larger sized clasts comprise ~50-60% of subunit, all subangular to subrounded medium gray quartzite up to 1' diameter, though mode average ~2-4"; poorly sorted; common plant and tree roots, especially in upper half; gradational basal contact.

2.2B Wasatch Fm (Tw): ~5.5-6' thick; moderate reddish brown (10R 4/6) to dark yellowish brown (10YR 4/2); sandy lean CLAY with gravel, stiff, slightly moist to moist, low plasticity, massive; gravel and larger sized clasts comprise ~40-50% of unit, all medium gray subrounded to subangular quartzite up to 1' diameter, though mode average 2-4"; poorly sorted; possible landslide deposit, as common angular clasts and variable clast sizes; occasional plant and tree roots; basal ~6"-1' is increasingly fat clay-rich; sharp, planar basal contact.

3. Paleosol 1: ~1.5' thick; dark reddish brown (10R 3/4) to moderate reddish brown (10R 4/6); fat CLAY, stiff, moist, high plasticity, massive; slickensides produced when excavating with hammer, but no natural slickensides observed; rare small rounded to subrounded quartzite clasts up to 1" diameter; occasional small plant roots and pinhole voids; irregular, gradational basal contact.

4. Paleosol 2: ~6"-1' thick; mottled moderate reddish brown (10R 4/6) and dark yellowish orange (10YR 6/6); silty, sandy fat CLAY, stiff, moist, high plasticity, massive, though retains some internal fine bedding and structure from original bedrock; unit is very highly, nearly totally weathered bedrock; few small plant roots.

5.Weathered Bedrock (Cn): At least ~1.5' thick; medium dark gray (N4) to moderate reddish brown (10R 4/6); unit contains clayey SAND and highly weathered dolomite bedrock clasts up to 4" diameter; where present, clay is fat; medium dense, moist to wet, massive; where present, clasts are very soft to soft and will crumble in hands.

FIGURE A-4
TP-3 LOG

SUMMIT HORIZON NEIGHBORHOOD

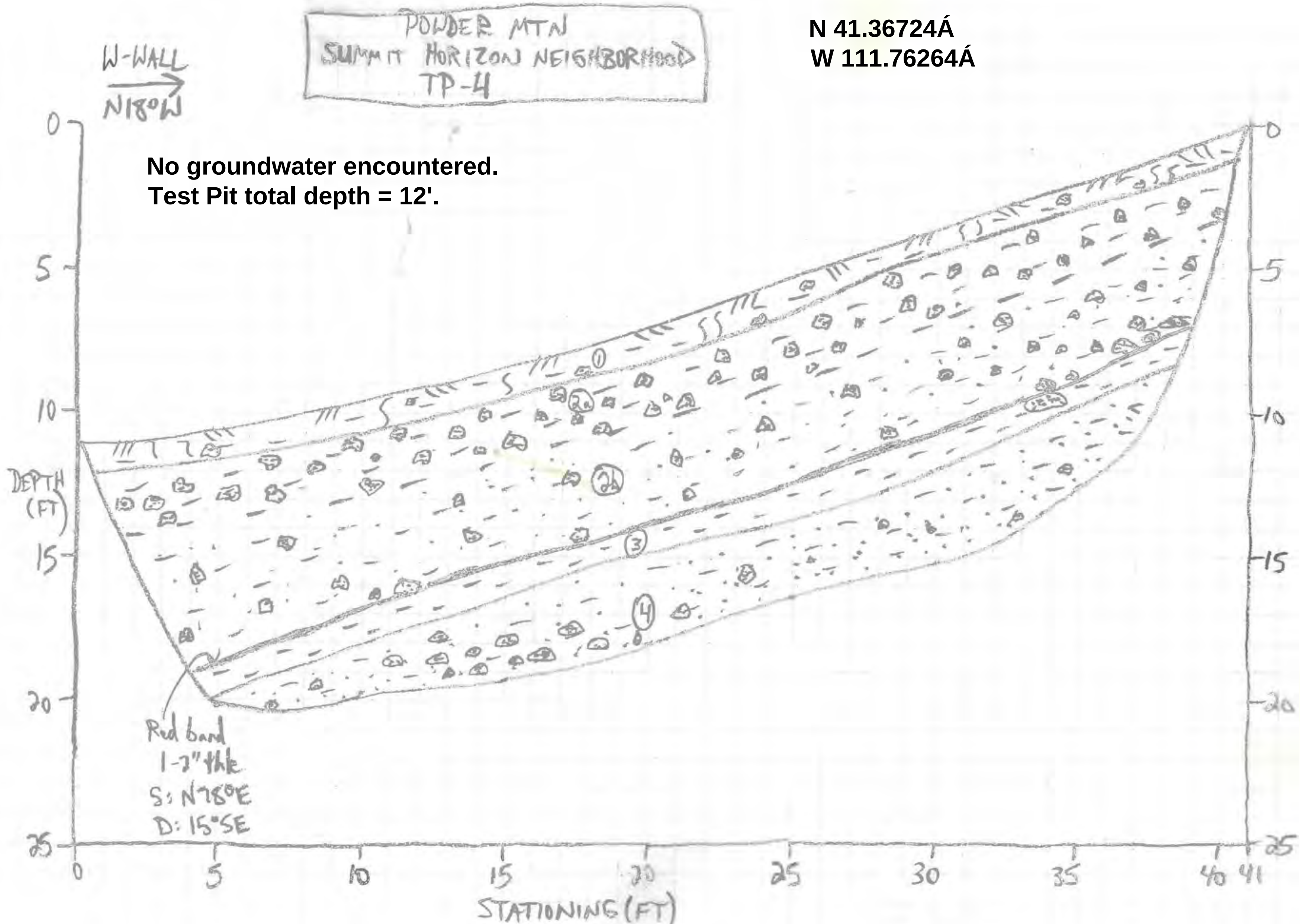
GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE:	IGES
PROJECT: 01628–013	1"=5'	

Project No. 01628-013

Date 6-8-16 by PED

Ckd by ______ on ______



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1' thick; dark yellowish brown (10YR 4/2) to grayish brown (5Y 3/2); lean CLAY, medium dense, slightly moist, low plasticity, massive; abundant plant and tree roots; occasional gravel and larger sized medium gray (N5) quartzite clasts up to 8" diameter.

2.2A Colluvium 1 (Qcl): ~2' thick; dark yellowish brown (10YR 4/2); lean CLAY with gravel gradational to clayey GRAVEL, medium stiff to loose, slightly moist, low plasticity, massive; minor silt; gravel and larger sized clasts comprise ~50-60% of subunit, all subangular to subrounded medium gray quartzite up to 1' diameter, though mode average ~6"; poorly sorted; common plant and tree roots, especially in upper half; gradational basal contact.

2.2B Colluvium 2 (Qcl): ~2.5-3' thick; dark yellowish brown (10YR 4/2) to moderate yellowish brown (10YR 5/4); lean CLAY with sand, stiff, slightly moist to moist, low plasticity, massive; gravel and larger sized clasts comprise ~40% of subunit, all subangular to subrounded medium gray quartzite up to 8" in diameter, though mode average 2-4"; poorly sorted; occasional plant and tree roots; sharp, largely planar basal contact; possibly Wasatch Formation?

3. Pond Clay?: ~1' thick; pale yellowish orange (10YR 6/2) to dark yellowish orange (10YR 6/6); fat CLAY, stiff, moist, moderate plasticity; thinly bedded at top, though largely massive; blocky texture; occasional subrounded to subangular quartzite clasts up to 5" in diameter, though commonly <1"; top of unit is 1-2" red band, though no evidence of shearing; no slickensides observed; sharp, planar basal contact.

4.Wasatch Fm (Tw): At least 3' thick; dark reddish brown (10R 3/4) to moderate reddish brown (10R 4/6); conglomeratic bedrock disaggregated into sandy CLAY gradational to clayey SAND with depth, medium stiff to medium dense, moist, massive, low plasticity; becomes sandier and more gravelly with depth; gravel and larger sized clasts comprise ~10-15% of unit, all subrounded to subangular quartzite up to 6" diameter, which is mode clast size though clasts are largely concentrated toward southern end of test pit; occasional plant and tree roots; due to low density and clast proportion, possibly another colluvial unit.

FIGURE A-5
TP-4 LOG

SUMMIT HORIZON NEIGHBORHOOD

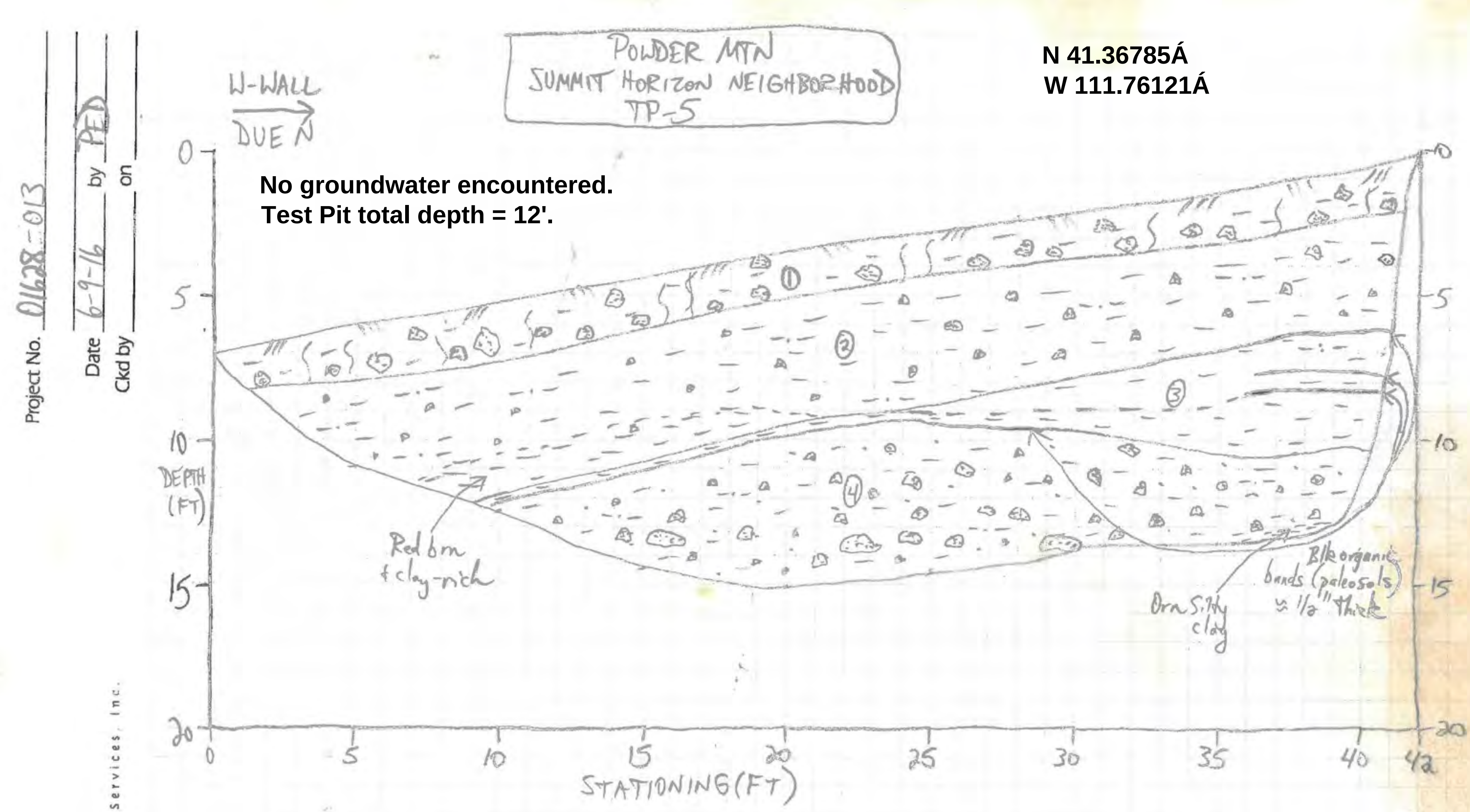
GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE: 1"=5'
PROJECT: 01628-013	

IGES Environmental Services, Inc.

N 41.36785Á
W 111.76121Á

No groundwater encountered.
Test Pit total depth = 12'.



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~2' thick; dark yellowish brown (10YR 4/2) to grayish brown (5Y 3/2); lean CLAY, medium dense, slightly moist, low plasticity, massive; abundant plant and tree roots; abundant gravel and larger sized medium gray (N5) quartzite clasts up to 1.5' diameter, with an 8" mode average; clasts comprise ~50% of unit..

2. Colluvium (Qcl):~4' thick; dark yellowish brown (10YR 4/2) to dark reddish brown 10R 3/4); silty lean CLAY with gravel, stiff, slightly moist, low plasticity, massive; gravel and larger clasts comprise ~20-25% of unit, up to 9" diameter; clasts are entirely subangular to subrounded medium gray (N5) to white (N9) quartzite, with mode average 2-4" diameter; becomes very clay-rich with fat clay to the south; poorly sorted; common plant and tree roots; sharp, planar basal contact.

3. Alluvial Fan?: ~6"-3.5' thick; pale yellowish orange (10YR 6/2); lean CLAY, stiff, slightly moist, low plasticity, massive, though includes subhorizontal thin paleosols; common pinhole voids throughout; thickens upslope and dips to south steeper than modern topography; occasional plant and tree roots; possibly representative of multiple debris-flow deposits.

4. Wasatch Fm? (Tw): At least ~6' thick; dark reddish brown (10R 3/4) to dark yellowish brown (10YR 4/2); silty, sandy lean CLAY with gravel, stiff, moist, low plasticity, massive to faintly bedded; gravel and larger sized clasts comprise ~50-60% of unit; clasts are entirely subrounded to subangular quartzite up to 1' diameter, though mode average 4-6"; occasional to common pinhole voids throughout; purely silty clay found in northernmost part of test pit; poorly sorted; occasional plant and tree roots.

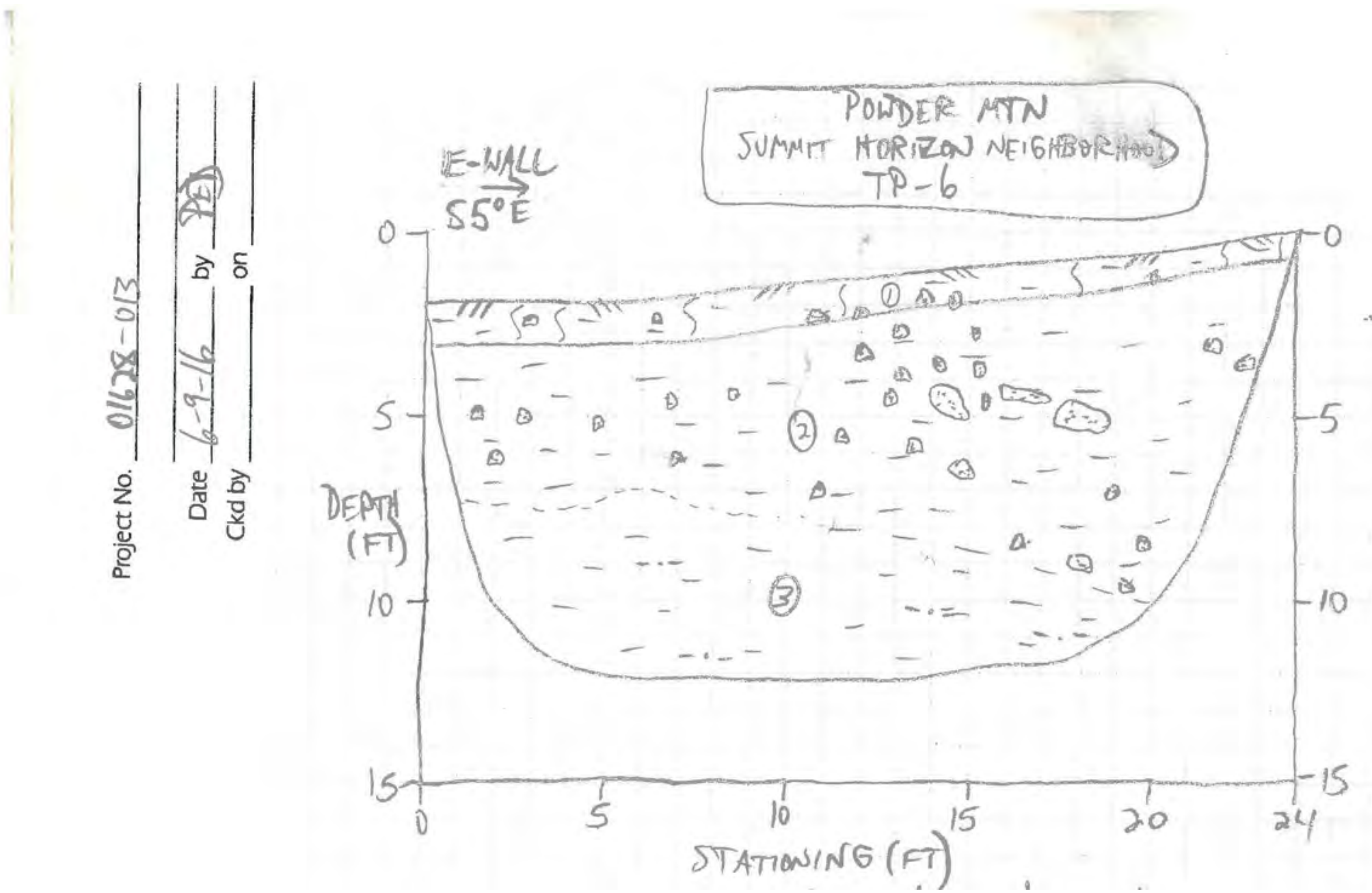
FIGURE A-6
TP-5 LOG

SUMMIT HORIZON NEIGHBORHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE:
PROJECT: 01628-013	1"=5'

IGES



N 41.36759Á
W 111.76068Á

Test Pit total depth = 11'.

At the time of logging (2:45 PM on 6-9-16), groundwater had filled test pit with a water column thickness of 2.15'.

LITHOLOGIC UNIT DESCRIPTIONS:

***All lithologic descriptions done from visual observation and photos; no access to test pit due to groundwater and sloughing.**

1. A/B Soil Horizon: ~1-1.5' thick; dark yellowish brown (10YR 4/2); lean CLAY, medium stiff, slightly moist, low plasticity, massive; some silt and occasional to common quartzite clasts up to 3" thick, though some boulders several feet in diameter noted on ground surface; clasts are subrounded to rounded, all white to pink quartzite; gravel and larger sized clasts comprise ~5% of unit; abundant plant and tree roots.

2. Wasatch Fm? (Tw): ~5' thick; moderate reddish brown (10R 4/6) to moderate red (5R 5/4); clayey GRAVEL with abundant (~75%) quartzite clasts up to 1.5' diameter, though mode average ~4"; clasts are subrounded to subangular; poorly sorted; occasional to few plant and tree roots; sharp, planar basal contact; possibly colluvium.

3. Pond Clay?: At least ~4' thick; medium dark gray (N4) to dark reddish brown (10R 3/4); silty fat CLAY, largely covered by groundwater; possible clasts; likely Wasatch Formation.

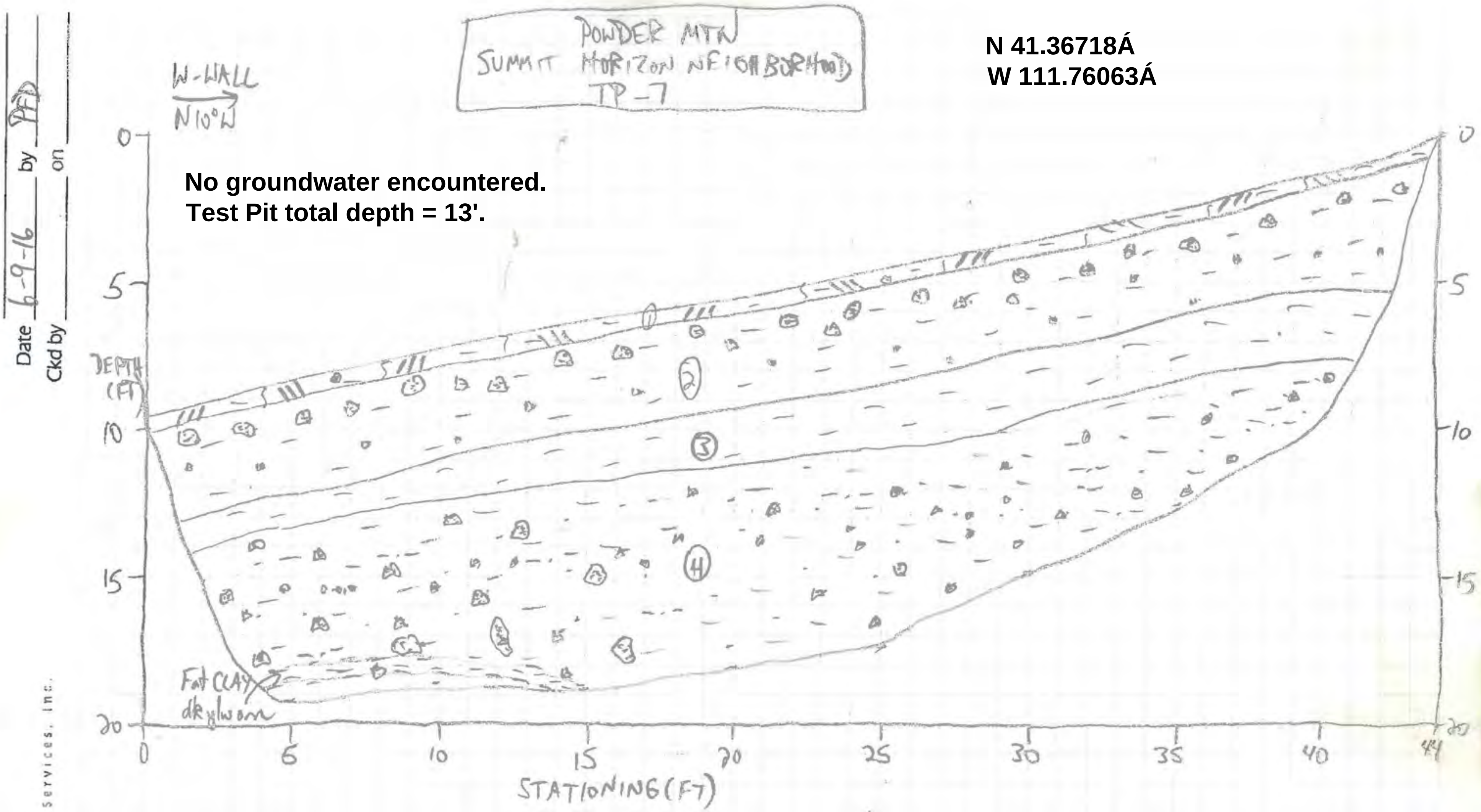
FIGURE A-7
TP-6 LOG

SUMMIT HORIZON NEIGHBORHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE:
PROJECT: 01628–013	1"=5'

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~6"-1' thick; dark yellowish brown (10YR 4/2) to grayish brown (5Y 3/2); lean CLAY, medium dense, slightly moist, low plasticity, massive; abundant plant and tree roots; abundant gravel and larger sized medium gray (N5) quartzite clasts up to 1.5' diameter, with an 8" mode average; clasts comprise ~50% of unit.

2. Colluvium (Qcl): ~3-3.5' thick; dark yellowish brown (10YR 4/2) to dark reddish brown (10R 3/4); lean CLAY with gravel, medium stiff, slightly moist, low plasticity, massive; reversely graded; gravel and larger sized clasts comprise ~20% of unit, but ~50% in upper 1/2 of unit; clasts all subrounded to subangular quartzite up to 10" diameter, though mode 4"; common plant and tree roots, especially in top 2'; becomes clayier with depth; sharp, planar basal contact.

3. Pond Clay?: ~1-2.5' thick; pale yellowish orange (10YR 6/2) to dark yellowish orange (10YR 6/6); fat CLAY, stiff, slightly moist, high plasticity, massive; common pinhole voids; occasional small plant roots; thickens to the north; sharp, planar basal contact.

4. Wasatch Fm (Tw): At least ~6' thick; dark reddish brown (10R 3/4) to pale yellowish orange (10YR 6/2); silty, sandy lean CLAY with gravel, stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~50-60% of unit, locally 70+%, entirely subrounded to subangular quartzite up to 1' diameter, though mode average 4-6"; poorly sorted; wet soil in places; contains fat clay subunit near base of the south side of the test pit.

FIGURE A-8
TP-7 LOG

SUMMIT HORIZON NEIGHBORHOOD

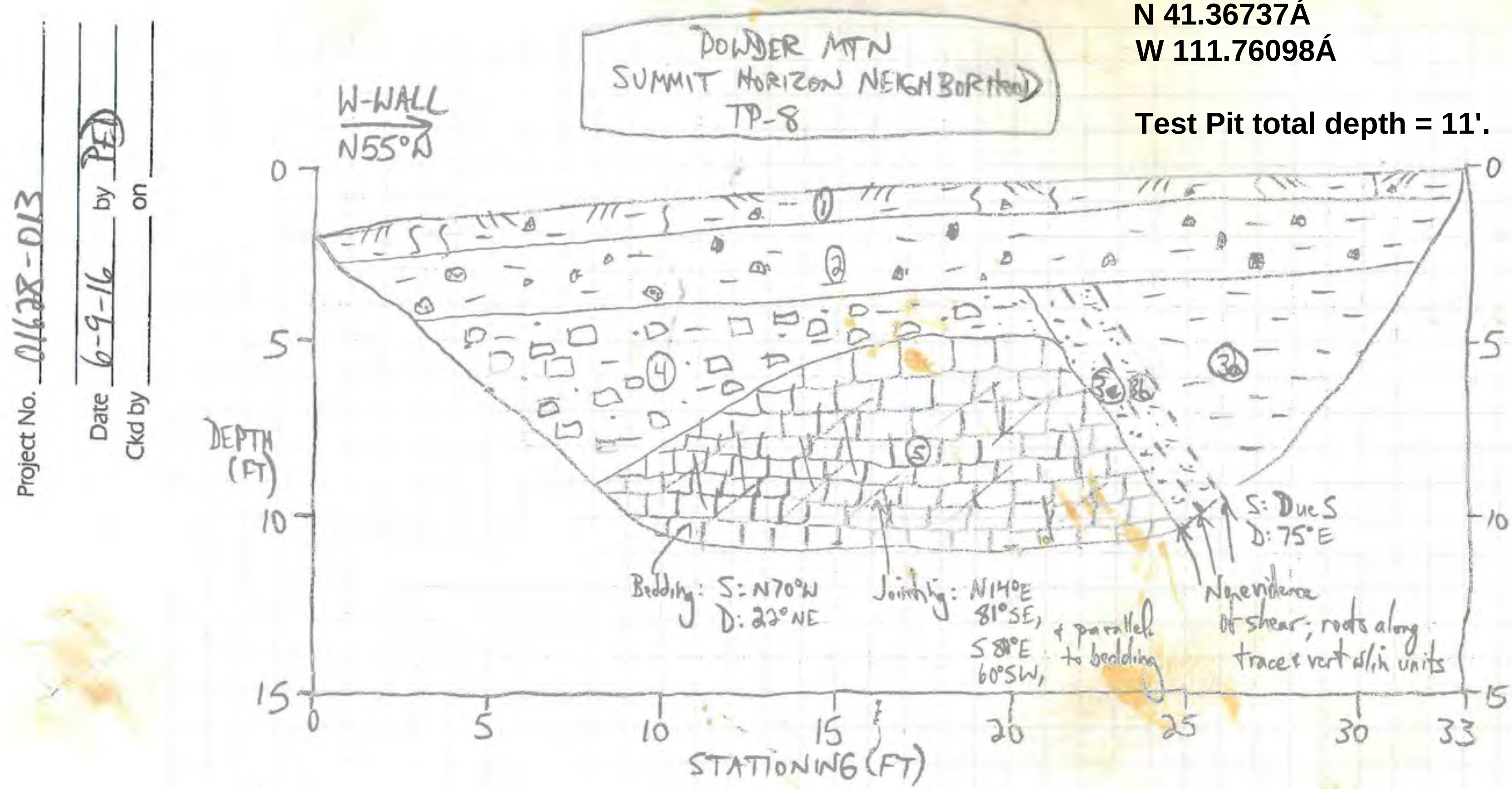
GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016 | SCALE: 1"=5'
PROJECT: 01628-013

IGES

N 41.36737Á
W 111.76098Á

Test Pit total depth = 11'.



Slow groundwater seepage from N wall of trench at 5' depth.

LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~6"-1' thick; dark yellowish brown (10YR 4/2) to grayish brown (5Y 3/2); lean CLAY, medium dense, slightly moist, low plasticity, massive; abundant plant and tree roots; abundant gravel and larger sized medium gray (N5) quartzite clasts up to 1.5' diameter, with an 8" mode average; clasts comprise ~50% of unit.

2. Colluvium (Qcl): ~2' thick; dark yellowish brown (10YR 4/2); lean CLAY with gravel, medium stiff, slightly moist, low plasticity, massive; gravel and larger sized clasts comprise ~15% of unit, and include subrounded to subangular quartzite and angular dark gray dolomite up to 6" diameter, though mode average 2-4"; likely B soil horizon with quartzite colluvial detritus; abundant plant and tree roots; sharp, planar basal contact.

3. Landslide (Qlso 2): ~5 thick, though rotated; contains 3 subunits:
- **3A**: Pale yellowish orange (10YR 6/2) fat CLAY, stiff, moist; common plant roots
- **3B**: Dark reddish brown (10YR 3/4) clayey SAND, medium dense, moist; common plant roots
- **3C**: Pale yellowish orange (10YR 6/2) silty SAND with clay, medium dense, wet; occasional plant roots

4. Weathered Bedrock (Cn): ~1.5-5' thick; grayish brown (5Y 3/2) to dark yellowish brown (10YR 4/2); clayey SAND with gravel, loose, moist, massive; abundant (>40%) angular (blocky) medium dark gray dolomite bedrock clasts up to 8" diameter, though mode average 4"; common plant and tree roots.

5. Bedrock (Cn): At least ~8' thick; medium dark gray finely sparry dolomite; thinly bedded (~1" spacing); well-developed blocky jointing; partially weathered.

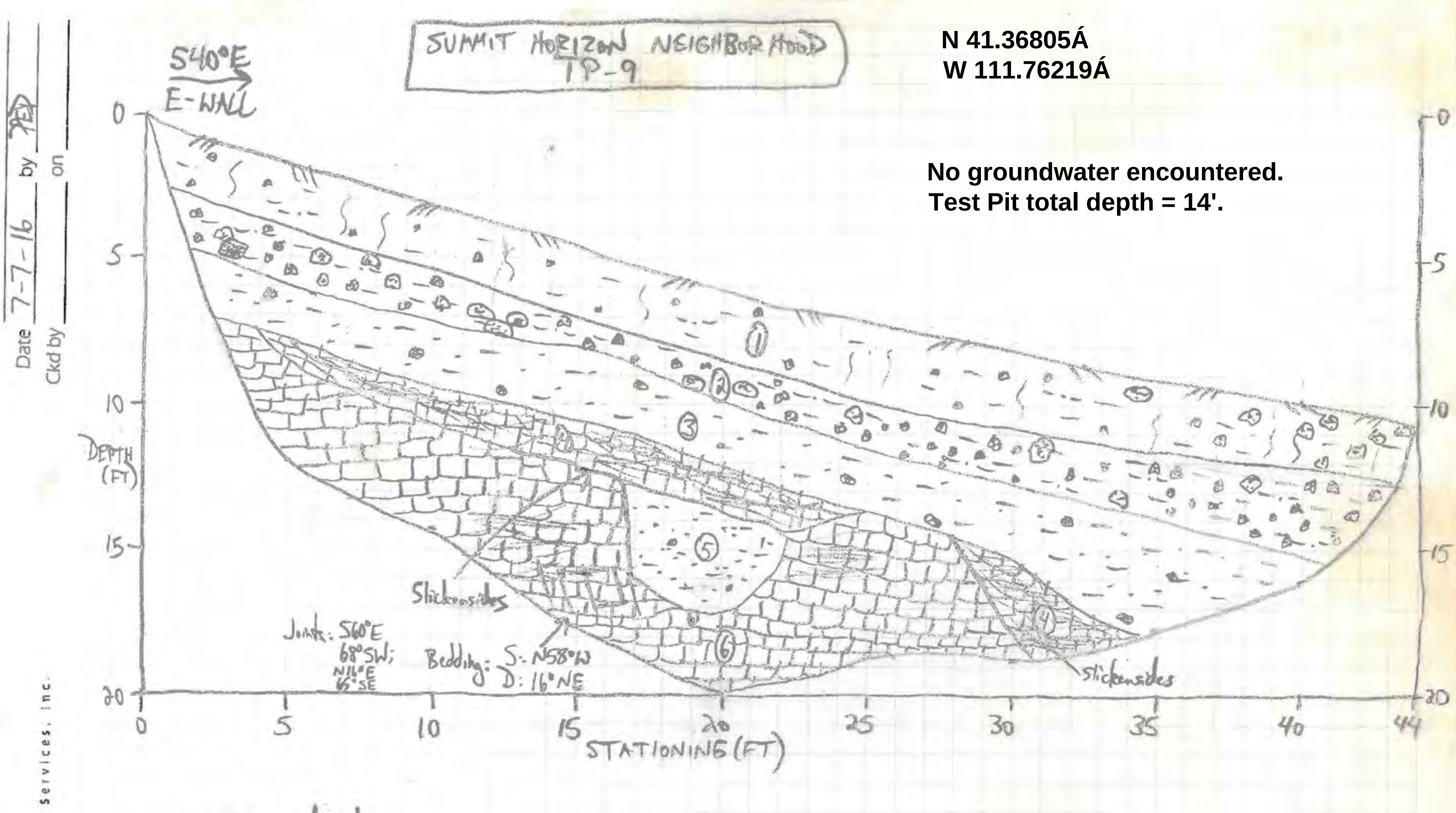
FIGURE A-9
TP-8 LOG

SUMMIT HORIZON NEIGHBORHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE:
PROJECT: 01628-013	1"=5'

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1.5-2' thick; dark yellowish brown (10YR 4/2); silty lean CLAY with gravel, medium stiff, dry, low plasticity, massive; gravel and larger sized clasts comprise ~15-20% of unit; clasts are entirely subrounded to rounded to subangular medium gray (N5) quartzite up to 1.5' diameter, though mode average <1" diameter; abundant plant and tree roots; planar, gradational basal contact.

2.Colluvium (Qcc/Qls): ~1-3 thick; light brown (5YR 6/4) to moderate yellowish brown (10YR 5/4); silty lean CLAY with gravel, gradational to clayey GRAVEL, medium stiff to stiff to medium dense, slightly moist to dry, low plasticity, massive; gravel and larger sized clasts comprise ~60% of unit, all subrounded to subangular light gray (N7) quartzite up to 1.5' diameter, though mode average 2-4"; poorly sorted; occasional to common pinhole voids (1-2 mm); abundant plant and tree roots; sharp, planar basal contact.

3. Paleosol: ~2' thick; dark reddish brown (10YR 3/4) to moderate reddish brown (10R 4/6); sandy fat CLAY, stiff to very stiff, slightly moist, moderate plasticity, massive; abundant pinhole voids throughout (1 mm); mottled in places near weathered bedrock; slickensides present on some surfaces consistent with modern slope (evidence of creep); occasional plant and tree roots; sharp, irregular basal contact.

4.Highly Weathered Bedrock (Cn): ~1-2' thick; pale yellowish orange (10YR 6/2) to medium gray (N5); dolomite bedrock largely decomposed into sandy fat CLAY, though relict bedding and bedrock clasts common and give mottled appearance; highly sandy in some areas; where present as clasts, bedrock shows thin bedding, though is very soft and crumbly; irregular, gradational basal contact.

5. Vug Infilling?: ~2.5' thick, localized; dark reddish brown (10R 3/4) to moderate reddish brown (10R 4/6); clayey SAND gradational to sandy fat CLAY; dense to stiff, moist, high plasticity, massive; sand almost entirely silica; rare (<5%) clasts up to 3" diameter of both dolomite and quartzite, evidence of vuggy infilling; some pinhole voids observed (1 mm); sharp, curved basal contact.

6.Bedrock (Cn): At least ~6' thick; medium gray (N5) to medium dark gray (N4) finely sparry dolomite unweathered, weathers to moderate reddish brown (10R 4/6) sandy and clayey surface; finely to medium bedded; most of exposed bedrock is largely weathered and medium hard, though both hard and soft parts can be found; despite extensive weathering, relict jointing is still present and easily discernible; slickensides observed on clayey weathering rind on both sides of bedrock outcrop (both upslope and downslope of bedrock), with slide direction indicating movement up and over bedrock.

FIGURE A-10

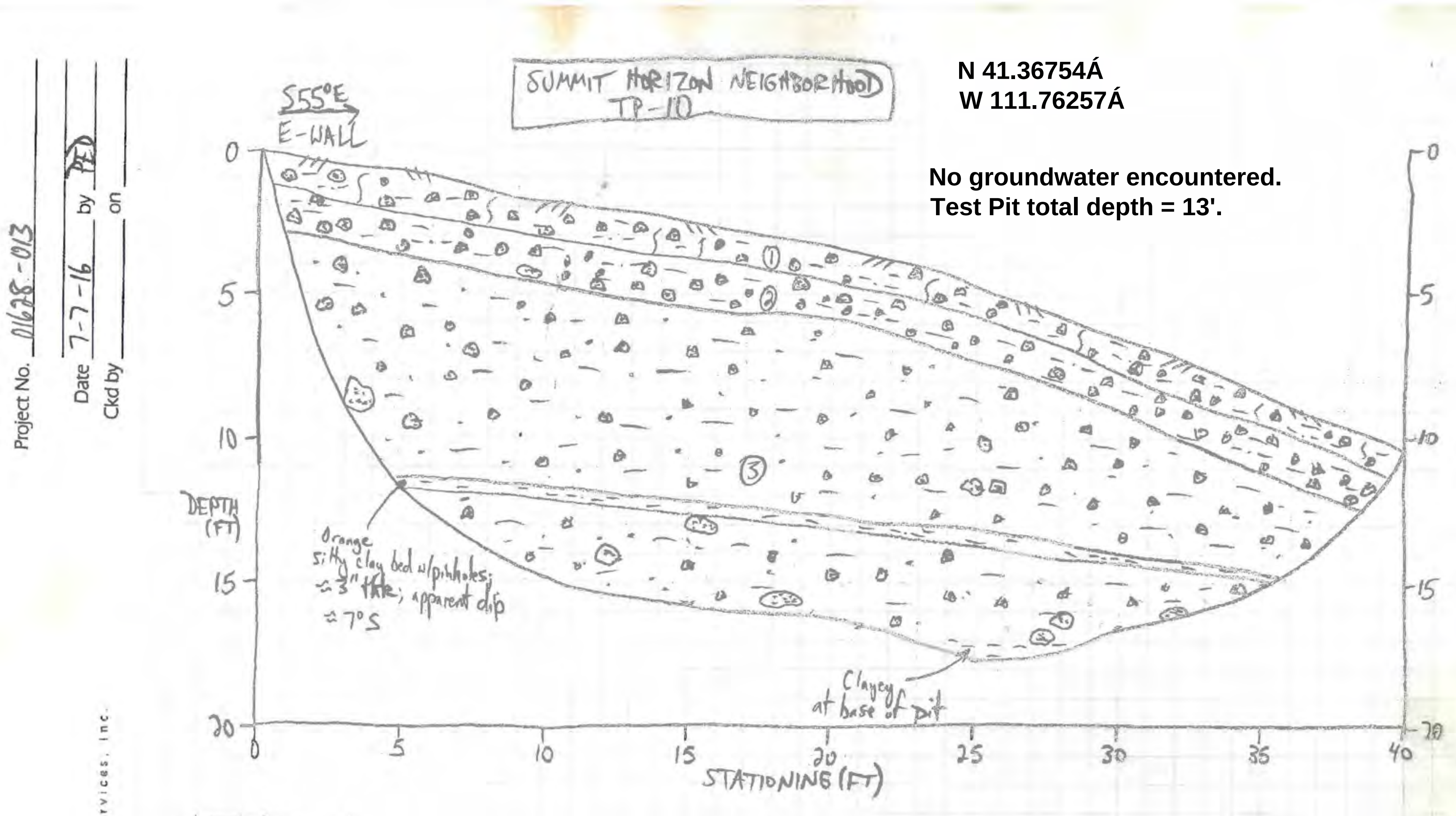
TP-9 LOG

SUMMIT HORIZON NEIGHBORHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016 | SCALE: 1"=5'
PROJECT: 01628-013

IGES



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1.5-2' thick; dark yellowish brown (10YR 4/2); silty lean CLAY with gravel, medium stiff, dry, low plasticity, massive; gravel and larger sized clasts comprise ~50-60% of unit; clasts are entirely subrounded to rounded to subangular medium gray (N5) quartzite up to 1.5' diameter, though mode average 3-4" diameter; abundant plant and tree roots; planar, gradational basal contact.

2. Colluvium (Qcc/Qls): ~1.5-2' thick; light brown (5YR 6/4) to moderate yellowish brown (10YR 5/4); silty lean CLAY with gravel, gradational to silty, clayey GRAVEL, stiff to very stiff to dense, slightly moist, low plasticity, massive; gravel and larger clasts comprise ~60% of unit, all subrounded to rounded to subangular quartzite up to 1' diameter, though mode average 3-4"; poorly sorted, but fairly well cemented; fine (1 mm) pinhole voids throughout; common plant and tree roots; irregular, gradational basal contact.

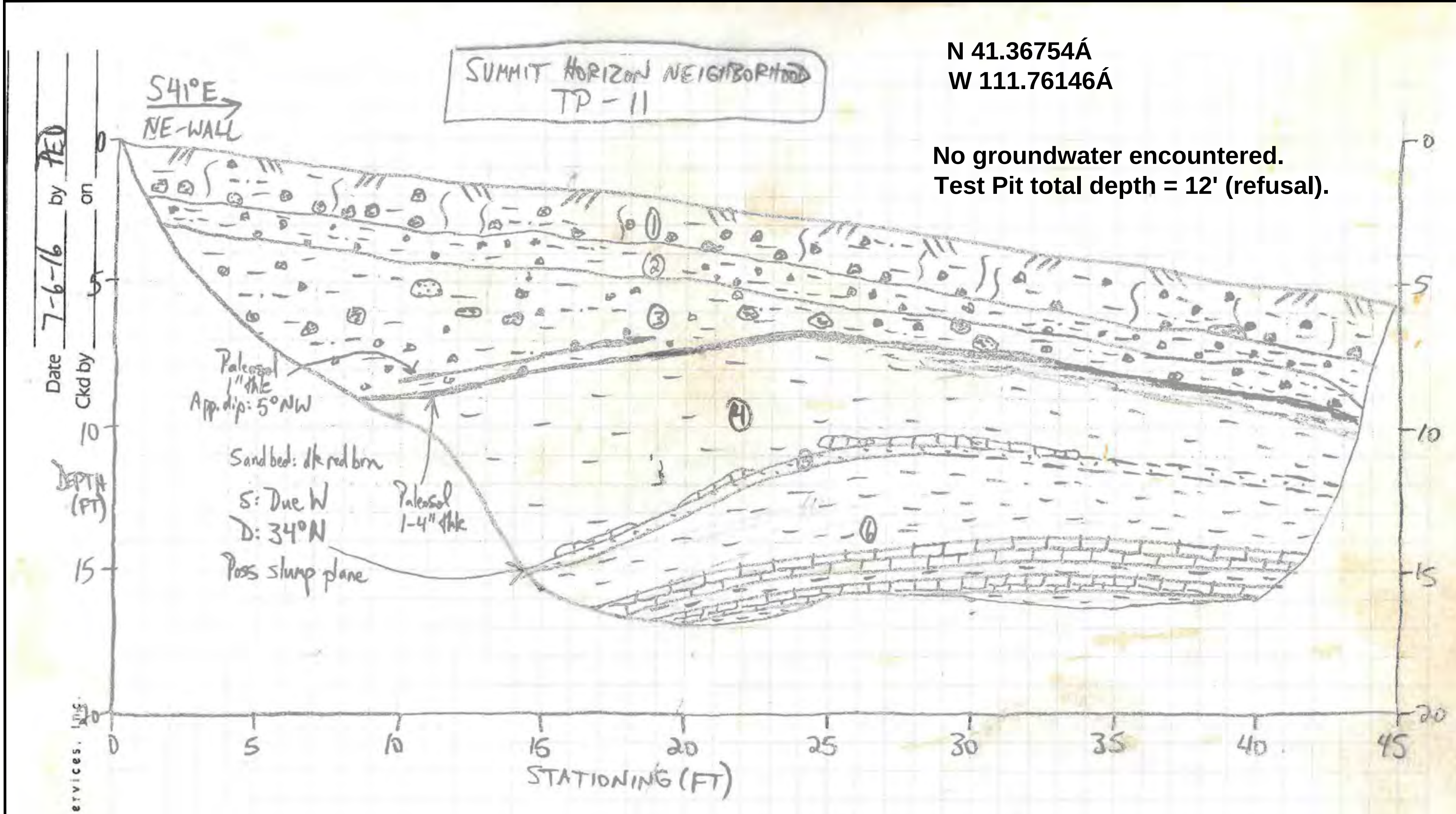
3. Wasatch Fm (Tw): At least ~10' thick; moderate reddish brown (10R 4/6) to dark yellowish brown (10YR 4/2); sandy CLAY with gravel, medium stiff to stiff, moist, low plasticity, massive; gravel and larger sized clasts comprise ~30-40% of unit, all subrounded to subangular quartzite up to 10" in diameter, though mode average 2-4"; abundant pinhole voids (1-2 mm) throughout; poorly sorted; common plant and tree roots; clay component increases with depth.

FIGURE A-11
TP-10 LOG

SUMMIT HORIZON NEIGHBORHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE:	IGES
PROJECT: 01628-013	1"=5'	



LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~1.5-2' thick; dark yellowish brown (10YR 4/2); silty lean CLAY with gravel, stiff, slightly moist, low plasticity, massive; gravel and larger sized clasts comprise ~35-40% of unit; clasts are entirely subrounded to subangular medium gray (N5) quartzite up to 1' diameter, though mode average 2-4" diameter; abundant plant and tree roots; planar, gradational basal contact.

2.Colluvium (Qcc/Qls): ~1-1.5' thick; light brown (5YR 6/4) to pale reddish brown (10R 5/4); silty lean CLAY with gravel, very stiff, dry, low plasticity, massive; well-cemented, as seen in TP-12; gravel and larger sized clasts comprise ~25% of unit, all subrounded to subangular quartzite up to 8" in diameter, though mode average 1-2"; occasional to common plant and tree roots; common pinhole voids (1 mm) throughout.

3.Wasatch Fm (Tw): ~1.5-4' thick; moderate reddish brown (10R 4/6) to dark reddish brown (10YR 3/4); silty lean CLAY with gravel, very stiff to stiff slightly moist, low plasticity, massive; fairly well cemented; blocky texture; gravel and larger sized clasts comprise ~30% of unit, all subrounded to subangular quartzite up to 1' diameter, though mode average 2-4"; clasts are more commonly larger than in the overlying colluvium unit; abundant pinhole voids (1-2 mm) throughout; occasional paleosols near base of unit; abundant plant and tree roots; thins dramatically to south; sharp, planar basal contact.

4. Alluvium: ~3-6' thick; pale yellowish orange (10YR 6/2) to very light gray (N8); fat CLAY, very stiff, dry, high plasticity, massive; extremely well cemented with some calcium carbonate; some possible relict shaley structure; occasional very thin plant roots; sharp, planar basal contact.

5.Weathered Bedrock 1 (Cn): ~6"-1' thick; dark reddish brown (10R 3/4); clayey SAND with highly weathered, thinly bedded dolomite blocks sitting within/right above sand bed; individual blocks are up to 1' diameter and are only partially continuous along unit; may possibly represent slump plane.

6.Weathered Bedrock 2 (Cn): At least ~5' thick; pale yellowish orange (10YR 6/2) to medium gray (N5); interbedded fat CLAY and partially weathered dolomite bedrock; clay is stiff to very stiff, moist, high to medium plasticity, thinly bedded; dolomite is finely sparry and thinly bedded, and weathers to a clinker-like appearance in places; clay beds have relict shale structure; individual dolomite beds are 6-8" thick, though western wall of test pit has much thicker beds and more continuous/less weathered bedrock due to dip.

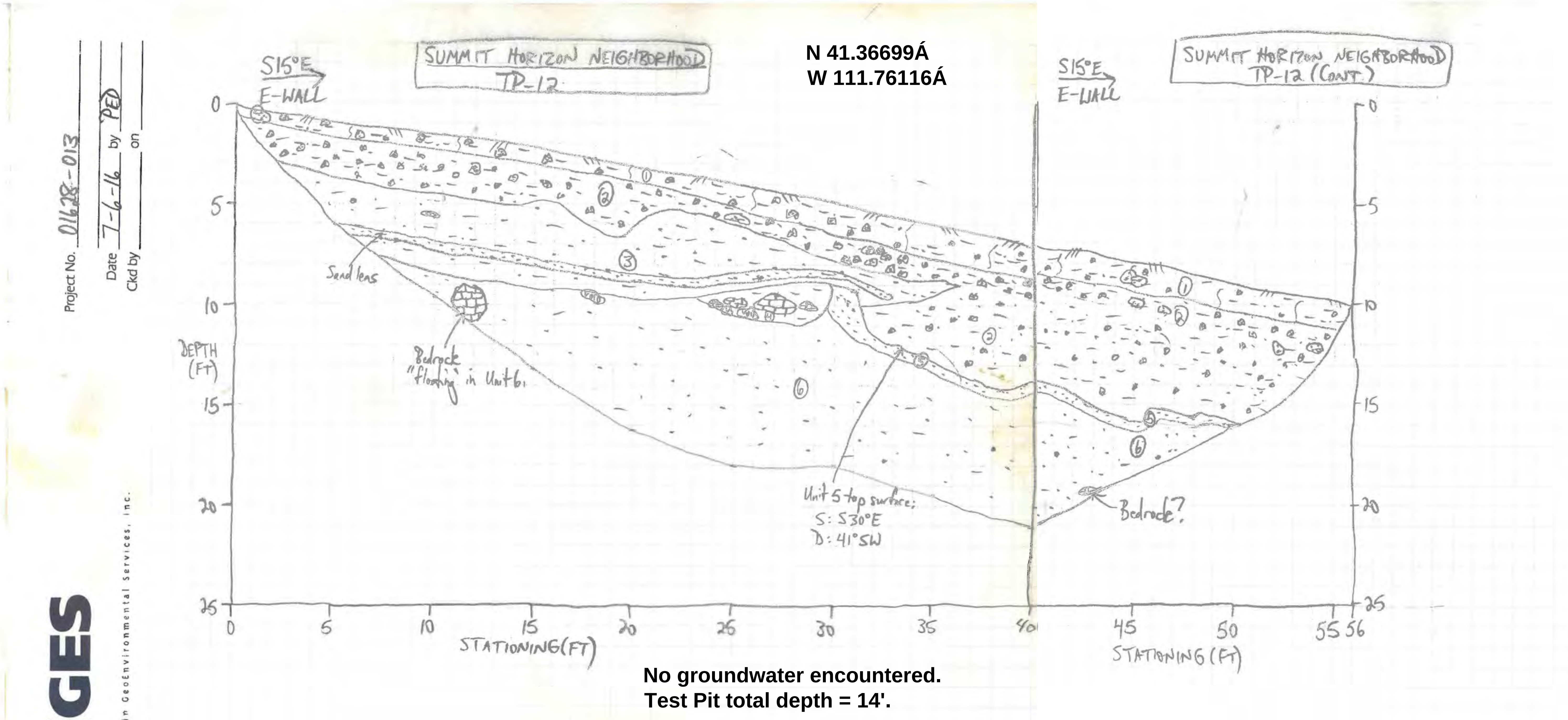
FIGURE A-12
TP-11 LOG

SUMMIT HORIZON NEIGHBORHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/18/2016	SCALE: 1"=5'
PROJECT: 01628-013	

IGES



No groundwater encountered.
Test Pit total depth = 14'.

LITHOLOGIC UNIT DESCRIPTIONS:

1. A/B Soil Horizon: ~6"-2' thick; dark yellowish brown (10YR 4/2); silty lean CLAY with gravel, medium stiff, dry, low plasticity, massive; gravel and larger sized clasts comprise ~35-40% of unit; clasts entirely pink to light gray quartzite, subrounded to subangular, up to 2.5' diameter, though mode average 1-2"; abundant plant and tree roots; gradational, irregular basal contact.

2. Colluvium (Qcc/Qls): ~2-6' thick; light brown (5YR 6/4) to light gray (N7) to dark yellowish brown (10YR 4/2); clayey SAND with gravel, very dense, slightly moist; very well cemented, though not calcareous; massive; moderately to poorly sorted; gravel and larger sized clasts comprise ~50-60% of unit, all subrounded to subangular quartzite up to 1' diameter, though mode average 2-4"; abundant pinhole voids (1-2 mm) throughout; thickens substantially to south (downslope); occasional plant and tree roots; sand component is very fine; sharp, irregular basal contact.

3. Alluvium 1: ~2-3' thick; pale yellowish orange (10YR 6/2) to moderate yellowish brown (10YR 5/4); clayey SAND, medium dense, slightly moist to moist, massive, though contains dark reddish brown (10YR 3/4) sand/sandstone lens 4" thick; abundant pinhole voids (1-2 mm) throughout; rare bedrock (weathered dolomite) clasts up to 5" diameter; sand component is very fine; abundant plant and tree roots; sharp, irregular basal contact.

4. Weathered Bedrock (Cn): ~1.5' thick; very dark gray (N2) to moderate reddish brown (10R 4/6); bedrock is dark gray to very dark gray, finely sparry dolomite, highly weathered, though still hard to very hard, massive; abundant associated fat clay weathering rinds and matrix within which blocks lie; multiple notable voids/cavities found below bedrock blocks; individual blocks up to 1.5' diameter; possibly landslide float.

5. Channel Sand: ~6"-1' thick; dark reddish brown (10R 3/4) to dark yellowish orange (10YR 6/6); fine, poorly graded SAND, medium dense to dense, moist; finely bedded, though kinked possibly due to creep or soft sediment deformation; minor silt and clay; highly irregular upper contact; occasional plant and tree roots.

6. Alluvium 2: At least 6' thick; dark reddish brown (10R 3/4) to moderate reddish brown (10R 4/6); sandy fat CLAY gradational to clayey SAND, medium stiff to stiff, slightly moist, moderate plasticity, massive; abundant pinhole voids throughout (up to 5 mm); occasional to rare (<5%) gravel sized or larger bedrock clasts (all angular dark gray dolomite) up to 3" diameter; rare plant and tree roots; becomes sandier with depth; possible bedrock at very bottom of pit at Station 43, though a void is present below the hard rocks.

FIGURE A-13
TP-12 LOG

SUMMIT HORIZON NEIGHBORHOOD
GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/19/2016 | SCALE: 1"=5'
FILE: 01628-013

IGES
IGES Intermountain GeoEnvironmental Services, Inc.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS	DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"	OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
LAYER	1/2 - 12"	FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNTRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

KEY TO SOIL SYMBOLS AND TERMINOLOGY

Project No. 01628-013

Engr. DAG

Drafted By DAG

Date July 2016

IGES

Intermountain Geo-Environmental Services, Inc.

Figure A-14

Rock Classification Should Include:

1. Rock name (or classification)
2. Color
3. Weathering
4. Fracturing
5. Competency
6. Additional comments indicating rock characteristics which might affect engineering properties

Weathering

Weathering	Field Test
Fresh	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	Slight discoloration inwards from open fractures, otherwise similar to Fresh.
Moderately Weathered	Discoloration throughout. Weaker minerals such as feldspar are decomposed. Strength somewhat less than fresh rock but cores cannot be broken by hand or scraped with a knife. Texture preserved.
Highly Weathered	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with a knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Completely Weathered	Minerals decomposed to soil but fabric and structure preserved. Specimens easily crumble or penetrated.

Fracturing

Spacing	Description
>6 ft	Very Widely
2-6 ft	Widely
8-24 in	Moderately
2 ½-8 in	Closely
¾-2 ½ in	Very Closely

Bedding of Sedimentary Rocks

Splitting Property	Thickness	Stratification
Massive	>4.0 ft	Very thick bedded
Blocky	2.0-4.0 ft	Thick-bedded
Slabby	2 ½-24 in	Thin-bedded
Flaggy	½-2 ½ in	Very thin-bedded
Shaly or platy	⅛ – ½ in	Laminated
Papery	< ⅛ in	Thinly laminated

RQD

RQD (%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

Competency

Class	Strength	Field Test	Approximate Range of Unconfined Compressive Strength (tsf)
I	Extremely Strong	Many blows with geologic hammer required to break intact specimen.	>2000
II	Very Strong	Hand-held specimen breaks with pick end of hammer under more than one blow.	2000-1000
III	Strong	Cannot by scraped or peeled with knife, hand-held specimen can be broken with single moderate blow with pick end of hammer	1000-500
IV	Moderately Strong	Can just be scraped or peeled with knife. Indentations 1-3 mm show in specimen with moderate blow with pick end of hammer.	500-250
V	Weak	Material crumbles under moderate blow with pick end of hammer and can be peeled with a knife, but is hard to hand-trim for triaxial test specimen.	250-10
VI	Friable	Material crumbles in hand.	N/A

KEY TO PHYSICAL ROCK PROPERTIES

Project No. 01628-013
Engr. DAG
Drafted By DAG
Date July 2016

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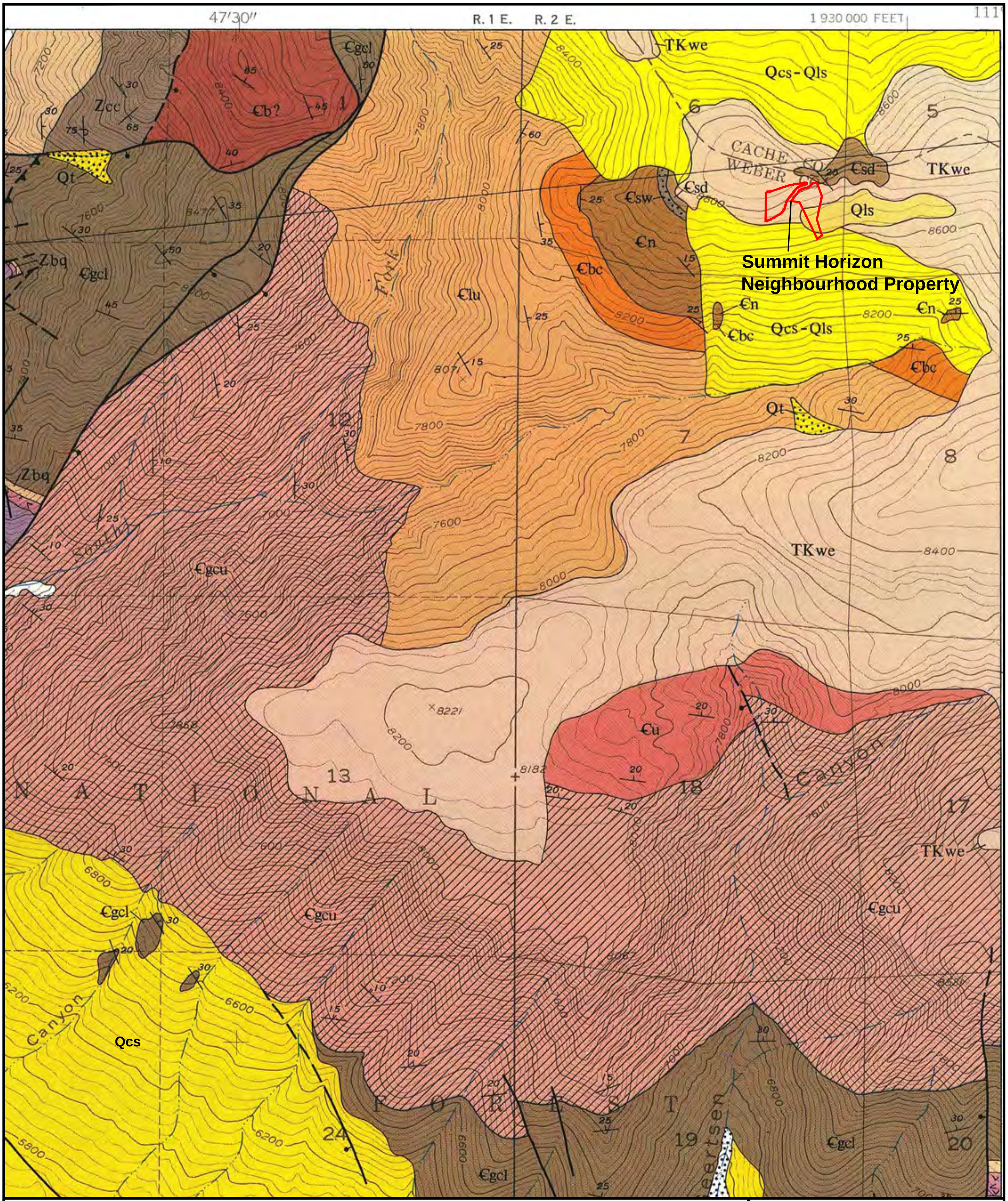
Figure A-15



BASE MAP:

USGS Huntsville 7.5-Minute Geologic Quadrangle Map (GQ-1503), Sorensen and Crittenden, Jr. (1979)

UTAH

QUADRANGLE LOCATION

0 1000 2000 3000

FEET

N

FIGURE A-16a

REGIONAL GEOLOGY MAP 1

SUMMIT HORIZON NEIGHBOURHOOD GEOLOGIC AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/22/2016
PROJECT:01628-013
SCALE: 1"=2,000'
IGES

MAP LEGEND

Qal ALLUVIAL DEPOSITS, UNDIFFERENTIATED (Holocene) – Unconsolidated gravel, sand, and silt deposits in presently active stream channels and floodplains; thickness 0-6 m

Qcs COLLUVIUM AND SLOPEWASH (Holocene) – Bouldery colluvium and slopewash chiefly along eastern margin of Ogden Valley; in part, lag from Tertiary units; thickness 0-30 m

Qf ALLUVIAL FAN DEPOSITS (Holocene) – Alluvial fan deposits; postdate, at least in part, time of highest stand of former Lake Bonneville; thickness 0-30 m

Qls LANDSLIDE DEPOSITS (Holocene) – thickness 0-6 m

Qt TALUS DEPOSITS (Holocene) – thickness 0-6 m

TKwe WASATCH AND EVANSTON(?) FORMATIONS, UNDIVIDED (Eocene, Paleocene, and Upper Cretaceous?) – Unconsolidated pale-reddish-brown pebble, cobble, and boulder conglomerate; forms boulder-covered slopes. Clasts are mainly Precambrian quartzite and are tan, gray, or purple; matrix is mainly poorly consolidated sand and silt; thickness 0-150 m

ST. CHARLES LIMESTONE (Upper Cambrian) – Includes:

Ꞓsd Dolomite member – Thin- to thick-bedded, finely to medium crystalline, light- to medium-gray, white- to light-gray-weathering, cliff-forming dolomite; linguloid brachiopods common in basal 15 m; thickness 150-245 m

Ꞓsw Worm Creek Quartzite Member – Thin-bedded, fine- to medium-grained, medium- to dark-gray, tan- to brown-weathering calcareous quartzitic sandstone; detrital grains well-sorted and well-rounded; thickness 6 m

Ꞓn NOUNAN DOLOMITE (Upper and Middle Cambrian) – Thin- to thick-bedded, finely crystalline, medium-gray, light- to medium-gray-weathering, cliff-forming dolomite; white twiggy structures common throughout unit; thickness 150-230 m

Ꞓbc CALLS FORT SHALE MEMBER OF BLOOMINGTON FORMATION (Middle Cambrian) – Olive-drab to light-brown shale and light- to dark-blue-gray limestone with intercalated orange to rusty-brown silty limestone; intraformational conglomerate common throughout unit; thickness 23-90 m

BASE MAP:
USGS Huntsville 7.5-Minute Geologic Quadrangle Map (GQ-1503), Sorensen and Crittenden, Jr. (1979)

QUADRANGLE LOCATION

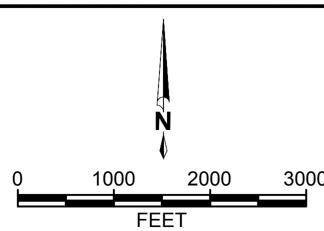


FIGURE A-16b

REGIONAL GEOLOGY MAP 1

SUMMIT HORIZON NEIGHBOURHOOD
GEOLOGIC AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/22/2016	SCALE: 1"=2,000'	IGES
PROJECT:01628-013		

MAP LEGEND

€lu — **CAMBRIAN LIMESTONES, UNDIVIDED** (Middle Cambrian) – Includes limestone and Hodges Shale Members of Bloomington Formation, and Blacksmith and Ute Limestones

€b — **BLACKSMITH LIMESTONE** (Middle Cambrian)) – Medium- to thin-bedded, light-gray to dark-blue-gray limestone; thin-bedded, flaggy-weathering, gray to tan silty limestone and interbedded siltstone; light- to dark-gray dolomite, with some reddish siliceous partings; thickness 400? m

€u — **UTE LIMESTONE** (Middle Cambrian) – Medium- to thin-bedded, finely crystalline, light- to dark-gray silty limestone with irregular wavy partings, mottled and streaked surfaces, worm tracks, and twiggy structures common throughout unit; oolites and *Girvanella* in many beds; olive-drab fissile shale interbedded throughout unit. Includes thin-bedded, gray-weathering, pale-tan to brown dolomite exposed at base of unit, 18-24 m at head of Geertsen Canyon and 0-3 m elsewhere; thickness 245? m

GEERTSEN CANYON QUARTZITE (Lower Cambrian) – Includes:

€gcu — Upper member – Pale-buff to white or flesh-pink quartzite, locally streaked with pale red or purple. Coarse-grained; small pebbles occur throughout unit and increase in abundance downward. Base marked by zone 30-60 m thick of cobble conglomerate in beds 30 cm to 2 m thick; clasts, 5-10 cm in diameter, are mainly reddish vein quartz or quartzite, sparse gray quartzite, or red jasper; thickness 730-820 m

€gcl — Lower member – Pale-buff to white and tan quartzite with irregular streaks and lenses of cobble conglomerate decreasing in abundance downward. Lower 90-120 m strongly arkosic, streaked greenish or pinkish. Feldspar clasts increase in size to 0.6-1.3 cm in lower part of unit; thickness 490-520 m

Recently active normal fault – Dashed where inferred. Ticks on downthrown side

Pre-Tertiary normal fault – Dotted where concealed Bar and ball on downthrown side

Thrust fault – Dashed where inferred Sawteeth on upper plate

BASE MAP:
USGS Huntsville 7.5-Minute Geologic Quadrangle Map (GQ-1503), Sorensen and Crittenden, Jr. (1979)

QUADRANGLE LOCATION

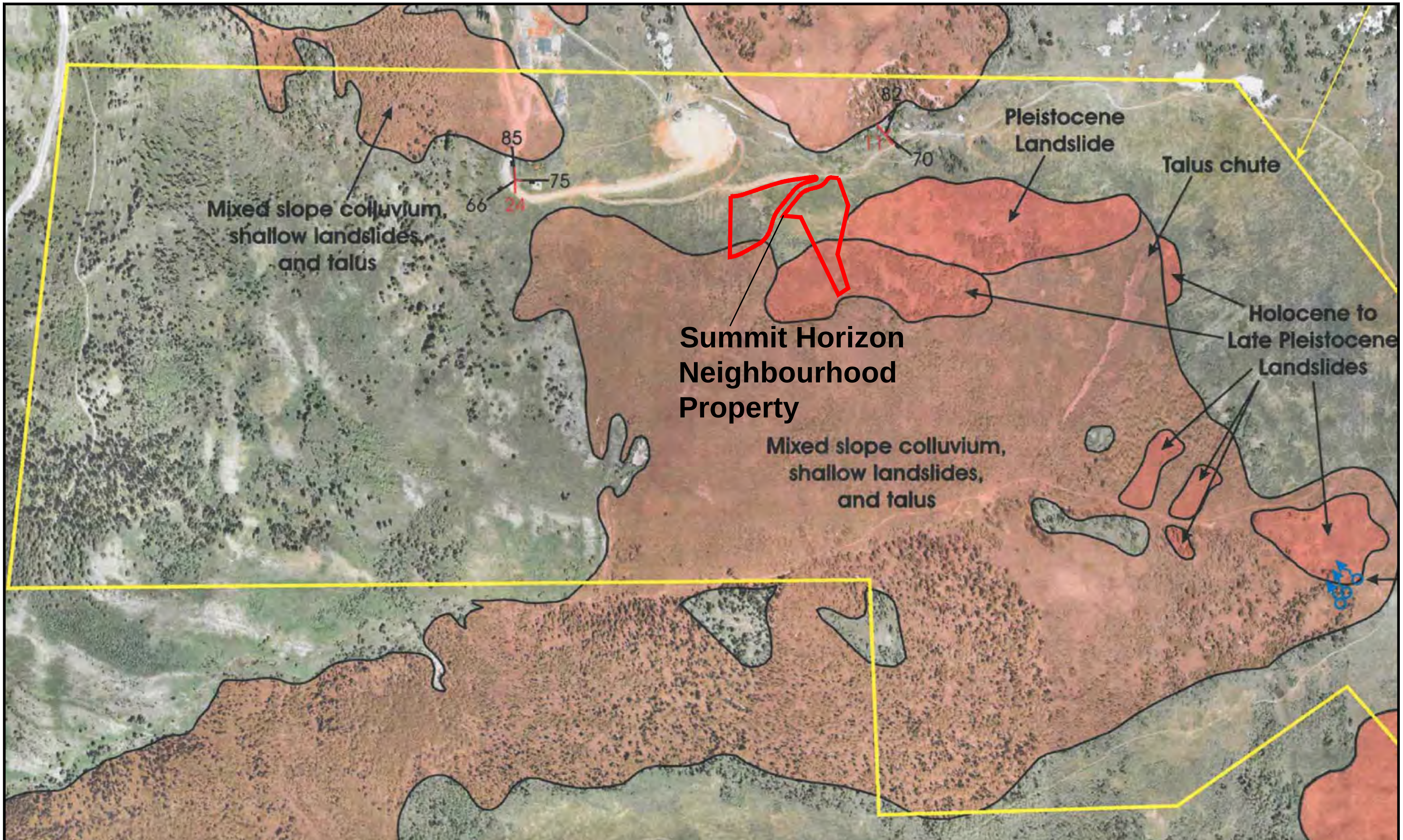
N

0 1000 2000 3000
FEET

FIGURE A-16c

REGIONAL GEOLOGY MAP 1
SUMMIT HORIZON NEIGHBOURHOOD
GEOLOGIC AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/22/2016
PROJECT:01628–013
SCALE: 1"=2,000'
IGES



BASE MAP:

Western Geologic (2012) Geologic Hazards Reconnaissance Report, Figure 3

FIGURE A-17

REGIONAL GEOLOGY MAP 2

SUMMIT HORIZON NEIGHBOURHOOD GEOLOGIC AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

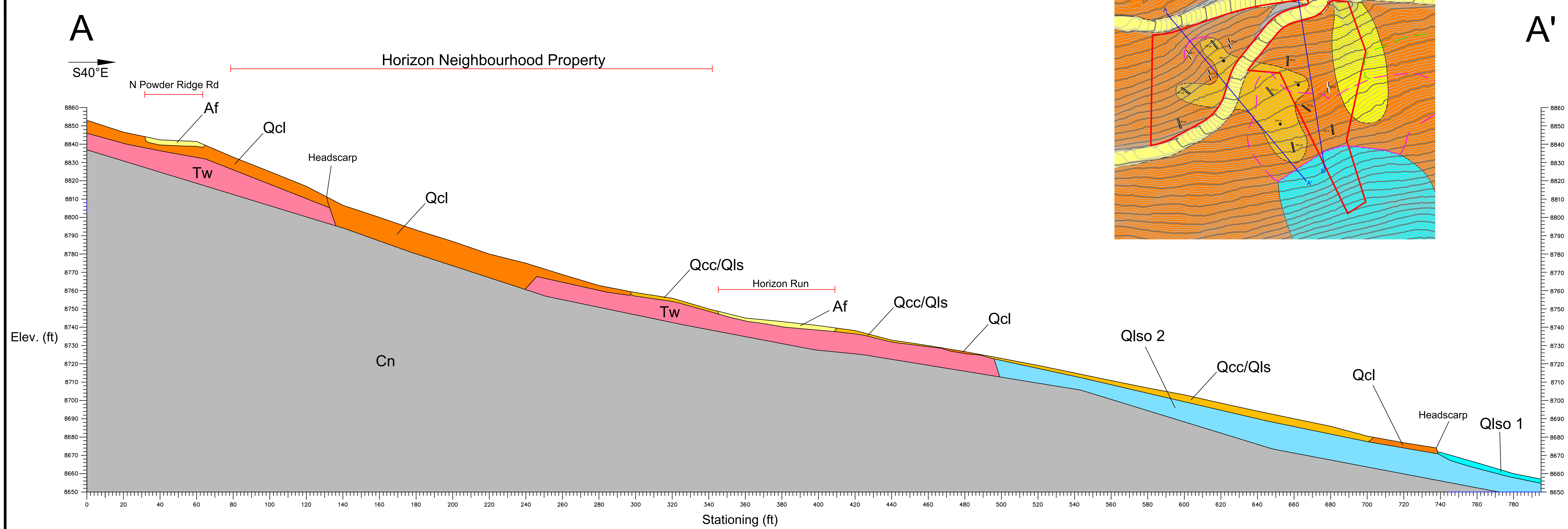
DATE: 07/22/2016
FILE: 01628-013
SCALE: 1"=1,000'
IGES

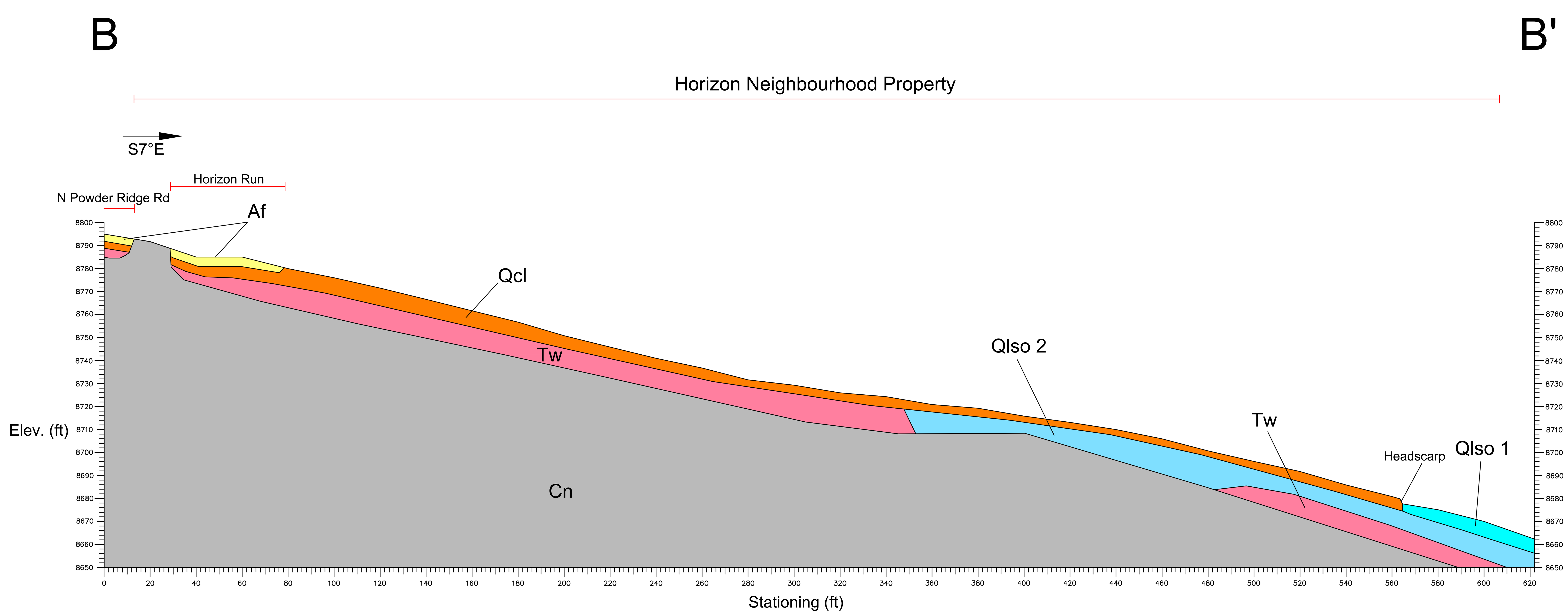
PLATE A-1
GEOTECHNICAL MAP

SUMMIT HORIZON NEIGHBOURHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/07/2017	SCALE: 1"=40'	
FILE: 01628-013 REV. 3		





LEGEND

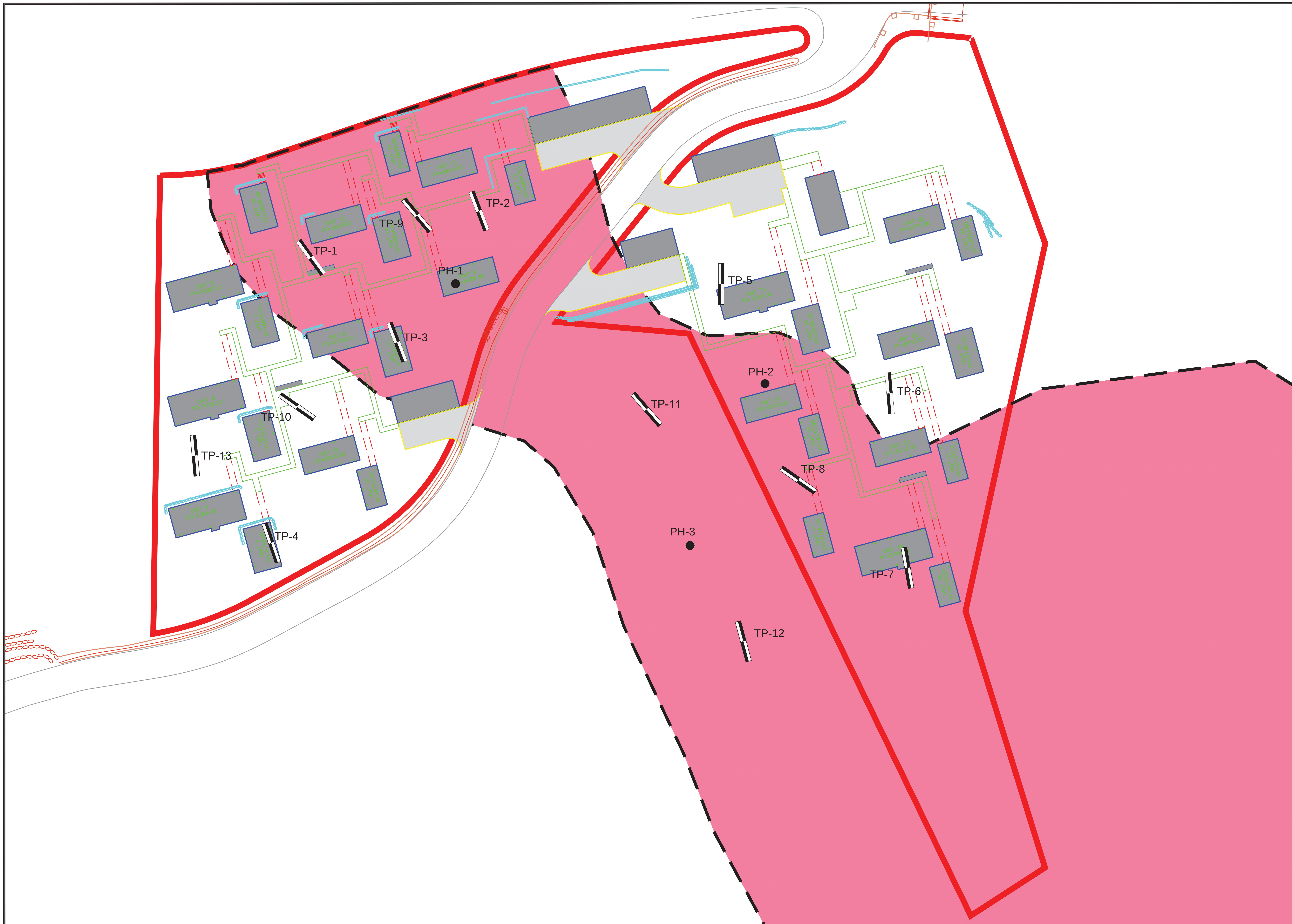
- HORIZON NEIGHBORHOOD PROPERTY BOUNDARY
- TP-10 TEST PIT LOCATION
- PH-3 POTHOLE LOCATION
- A — A' CROSS-SECTION LINE
- LANDSLIDE HEADSCARP
- APPROXIMATE SUBSURFACE LATE PLEISTOCENE? LANDSLIDE TRACE
- APPROXIMATE PLEISTOCENE LANDSLIDE TRACE (WESTERN GEOLOGIC, 2012)
- Afc: CURRENT CONSTRUCTION
- Af: ARTIFICAL FILL (ROADS)
- Qcl: LOOSE COLLUVIUM
- Qcc/Qls: CEMENTED LANDSLIDE COLLUVIUM
- Qcb: BEDROCK COLLUVIUM
- Qlso 1: PLEISTOCENE LANDSLIDE
- Qlso 2: PLEISTOCENE LANDSLIDE (In X-Section Only)
- Tw: WASATCH FORMATION (In X-Section Only)
- Cn: NOUNAN DOLOMITE

0 30' 60'
1" = 30'

PLATE A-2
CROSS-SECTIONS

SUMMIT HORIZON NEIGHBOURHOOD
GEOLOGIC HAZARD AND GEOTECH STUDY
POWDER MOUNTAIN RESORT
WEBER COUNTY, UTAH

DATE: 07/26/2016
FILE: 01628-013
REV. 1
SCALE: 1"=30'
IGES



LEGEND

HORIZON NEIGHBORHOOD PROPERTY BOUNDARY

TP-10 TEST PIT LOCATION

PH-3 POTHOLE LOCATION

APPROXIMATE MASS-MOVEMENT (SOIL CREEP AND SHALLOW LANDSLIDE) AFFECTED AREA

*STRUCTURES WITHIN OR STRADDLING SHADED AREA MUST BE SUPPORTED ON DRILLED PIER FOUNDATIONS ANCHORED 10 FEET INTO DOLOMITE BEDROCK.

STRUCTURES OUTSIDE OF THE BOUNDARY ARE EXPECTED TO BE SUPPORTED ON DRILLED PIER FOUNDATIONS ANCHORED 10 FEET INTO WASATCH FORMATION BEDROCK OR DOLOMITE BEDROCK, IF PRESENT LESS THAN 20 FEET BELOW EXISTING GRADE.

N

0 40' 80'

1" = 40'

PLATE A-3

FOOTINGS DELINEATION MAP

SUMMIT HORIZON NEIGHBORHOOD

GEOLOGIC HAZARD AND GEOTECH STUDY

POWDER MOUNTAIN RESORT

WEBER COUNTY, UTAH

DATE: 06/09/2017
FILE: 01628-013
REV. 2

SCALE: 1"=40'

IGES

APPENDIX B

Water Content and Unit Weight of Soil

(In General Accordance with ASTM D7263 Method B and D2216)

IGES

Project: **Summit/Horizon Neighborhood**
No: **01628-013**
Location: Eden, UT
Date: 7/19/2016
By: IM

Sample Info.	Boring No.	TP-1	TP-3	TP-4	TP-4	TP-5	TP-7	TP-8	
	Sample								
	Depth	13.0'	7.0'	5.0'	7.0'	2.0'	2.0'	8.0'	
	Split	No	Yes	Yes	No	Yes	Yes	Yes	
	Split sieve		3/4"	3/4"		3/8"	3/4"	3/4"	
	Total sample (g)		4562.64	4471.61		4506.59	3728.40	3454.16	
	Moist coarse fraction (g)		1499.35	1656.24		2808.81	819.59	18.75	
	Moist split fraction (g)		3063.29	2815.37		1697.78	2908.81	3435.41	
Unit Weight Data	Sample height, H (in)				5.075				
	Sample diameter, D (in)				2.416				
	Mass rings + wet soil (g)				1041.74				
	Mass rings/tare (g)				253.69				
	Moist unit wt., γ_m (pcf)				129.0				
Coarse Fraction	Wet soil + tare (g)		2504.16	2425.67		3119.30	1130.55	140.62	
	Dry soil + tare (g)		2464.49	2363.01		3071.09	1116.09	137.28	
	Tare (g)		711.53	408.18		310.49	310.96	121.87	
	Water content (%)		2.3	3.2		1.7	1.8	21.7	
Split Fraction	Wet soil + tare (g)	258.12	1911.57	1827.52	555.43	341.30	1737.24	1844.47	
	Dry soil + tare (g)	227.37	1801.25	1719.46	494.40	301.80	1552.15	1650.78	
	Tare (g)	120.72	316.54	331.47	128.49	122.00	333.12	409.03	
	Water content (%)	28.8	7.4	7.8	16.7	22.0	15.2	15.6	
	Water Content, w (%)	**28.8**	**5.7**	**6.0**	**16.7**	**8.5**	**11.9**	**15.6**	
	Dry Unit Wt., γ_d (pcf)				**110.6**				

Entered by:___________
Reviewed:___________

Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/18/2016
By: BRR

Boring No.: TP-1
Sample:
Depth: 13.0'
Description: Reddish brown fat clay

Preparation method: Wet
Liquid limit test method: Multipoint

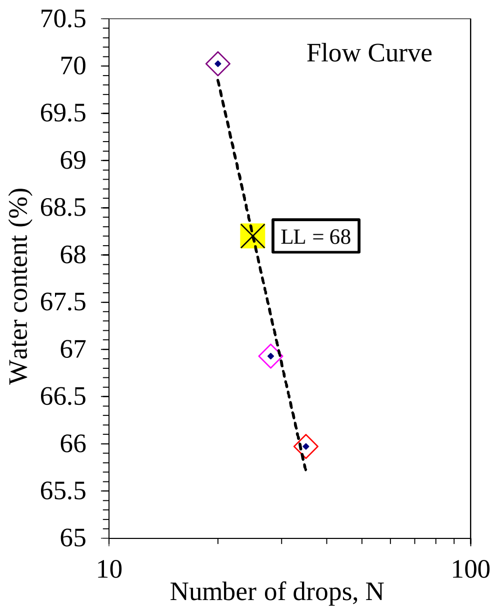
Plastic Limit

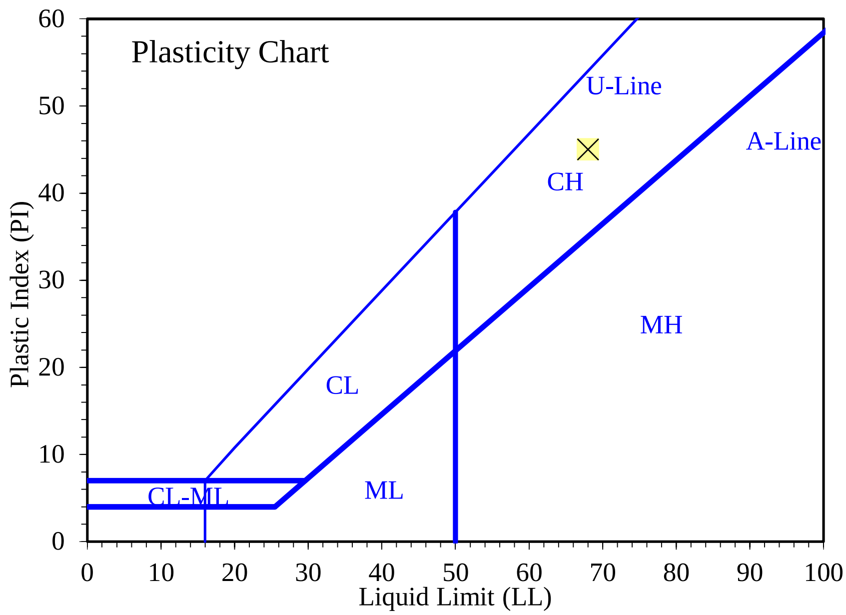
Determination No	1	2				
Wet Soil + Tare (g)	28.55	28.72				
Dry Soil + Tare (g)	27.31	27.46				
Water Loss (g)	1.24	1.26				
Tare (g)	22.03	22.06				
Dry Soil (g)	5.28	5.40				
Water Content, w (%)	23.48	23.33				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	35	28	20			
Wet Soil + Tare (g)	28.25	30.44	28.87			
Dry Soil + Tare (g)	25.71	27.02	26.02			
Water Loss (g)	2.54	3.42	2.85			
Tare (g)	21.86	21.91	21.95			
Dry Soil (g)	3.85	5.11	4.07			
Water Content, w (%)	65.97	66.93	70.02			
One-Point LL (%)		68	68			

Liquid Limit, LL (%)	**68**
Plastic Limit, PL (%)	**23**
Plasticity Index, PI (%)	**45**





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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BRR

Boring No.: TP-2
Sample:
Depth: 5.0'
Description: Reddish brown fat clay

Preparation method: Wet
Liquid limit test method: Multipoint

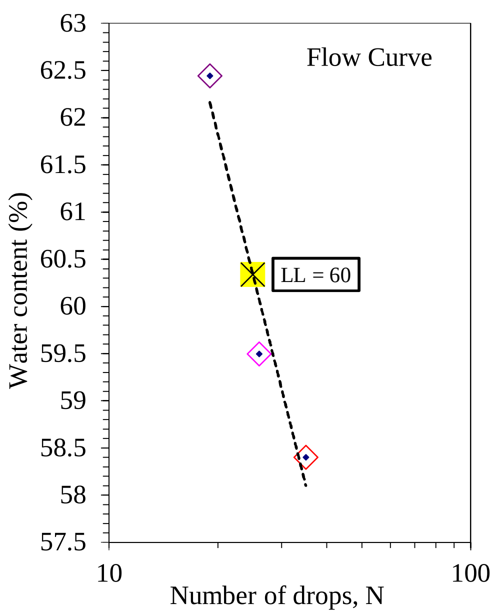
Plastic Limit

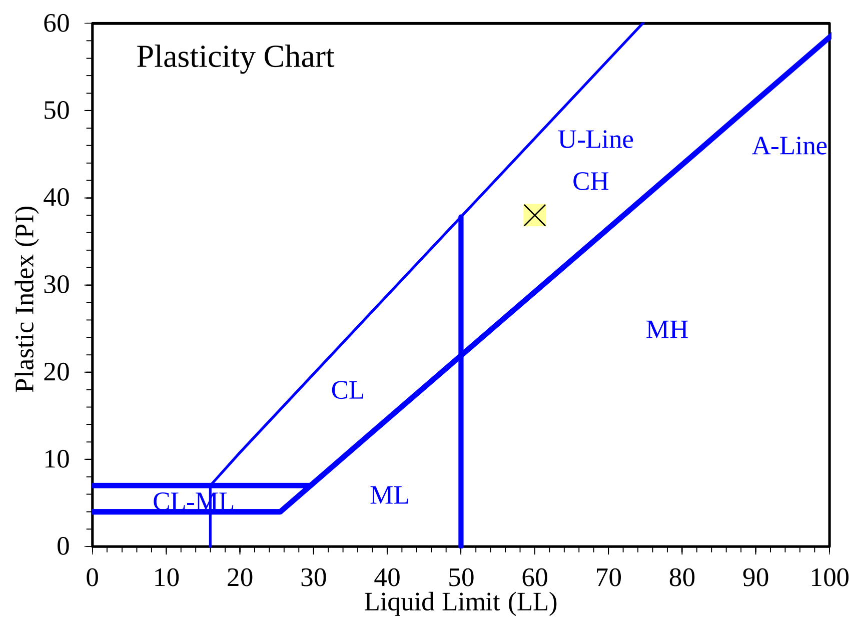
Determination No	1	2				
Wet Soil + Tare (g)	28.65	27.95				
Dry Soil + Tare (g)	27.50	26.83				
Water Loss (g)	1.15	1.12				
Tare (g)	22.15	21.69				
Dry Soil (g)	5.35	5.14				
Water Content, w (%)	21.50	21.79				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	35	26	19			
Wet Soil + Tare (g)	30.02	29.04	29.35			
Dry Soil + Tare (g)	27.10	26.44	26.54			
Water Loss (g)	2.92	2.60	2.81			
Tare (g)	22.10	22.07	22.04			
Dry Soil (g)	5.00	4.37	4.50			
Water Content, w (%)	58.40	59.50	62.44			
One-Point LL (%)		60				

Liquid Limit, LL (%)	**60**
Plastic Limit, PL (%)	**22**
Plasticity Index, PI (%)	**38**





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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

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Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BRR

Boring No.: TP-3
Sample:
Depth: 7.0'
Description: Reddish brown silty clay

Preparation method: Wet
Liquid limit test method: Multipoint

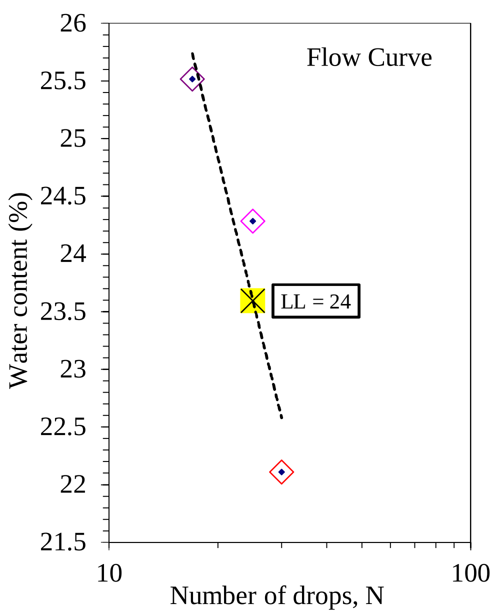
Plastic Limit

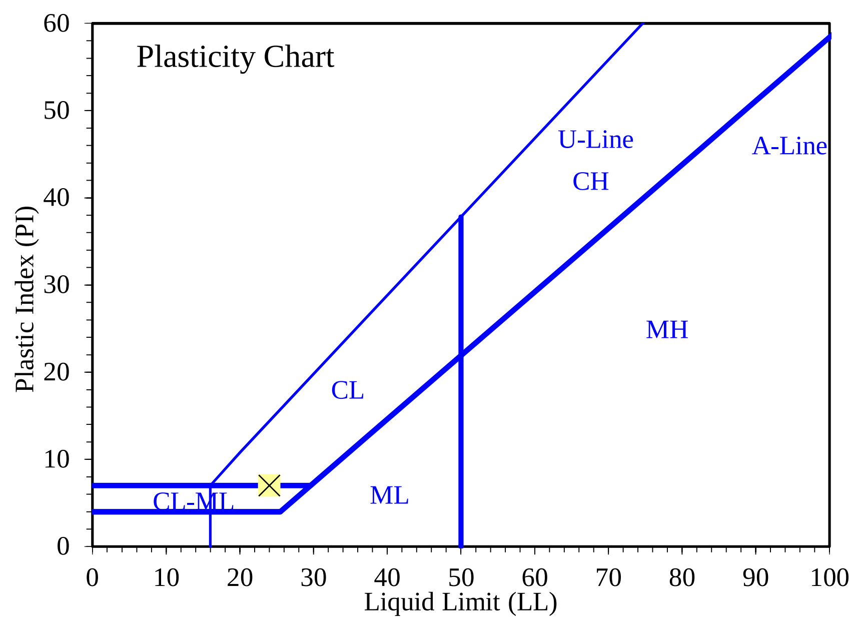
Determination No	1	2				
Wet Soil + Tare (g)	29.45	28.92				
Dry Soil + Tare (g)	28.38	27.92				
Water Loss (g)	1.07	1.00				
Tare (g)	22.26	21.93				
Dry Soil (g)	6.12	5.99				
Water Content, w (%)	17.48	16.69				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	30	25	17			
Wet Soil + Tare (g)	31.61	29.90	30.68			
Dry Soil + Tare (g)	29.87	28.37	28.95			
Water Loss (g)	1.74	1.53	1.73			
Tare (g)	22.00	22.07	22.17			
Dry Soil (g)	7.87	6.30	6.78			
Water Content, w (%)	22.11	24.29	25.52			
One-Point LL (%)	23	24				

Liquid Limit, LL (%)	**24**
Plastic Limit, PL (%)	**17**
Plasticity Index, PI (%)	**7**





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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BRR

Boring No.: TP-4
Sample:
Depth: 7.0'
Description: Light brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

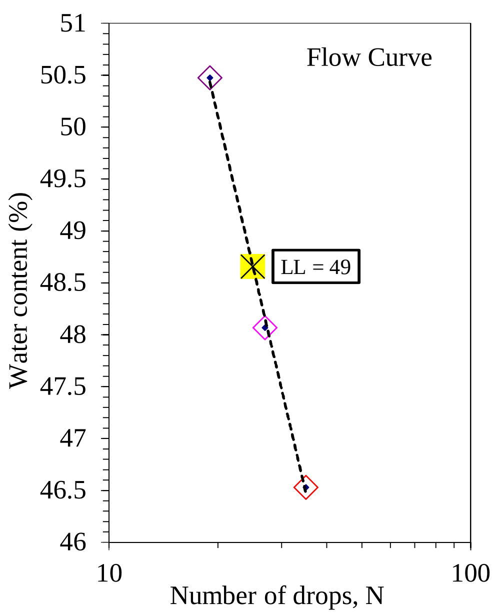
Plastic Limit

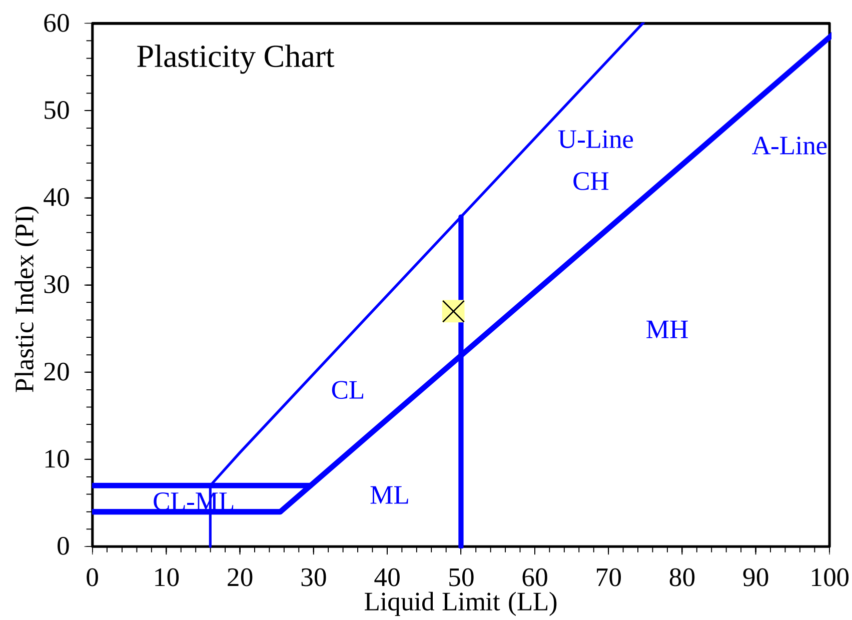
Determination No	1	2				
Wet Soil + Tare (g)	28.22	28.48				
Dry Soil + Tare (g)	27.14	27.32				
Water Loss (g)	1.08	1.16				
Tare (g)	22.11	22.00				
Dry Soil (g)	5.03	5.32				
Water Content, w (%)	21.47	21.80				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	35	27	19			
Wet Soil + Tare (g)	31.51	28.24	29.86			
Dry Soil + Tare (g)	28.56	26.00	27.20			
Water Loss (g)	2.95	2.24	2.66			
Tare (g)	22.22	21.34	21.93			
Dry Soil (g)	6.34	4.66	5.27			
Water Content, w (%)	46.53	48.07	50.47			
One-Point LL (%)		49				

Liquid Limit, LL (%)	**49**
Plastic Limit, PL (%)	**22**
Plasticity Index, PI (%)	**27**





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Liquid Limit, Plastic Limit, and Plasticity Index of Soils
(ASTM D4318)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BRR

Boring No.: TP-5
Sample:
Depth: 2.0'
Description: Brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

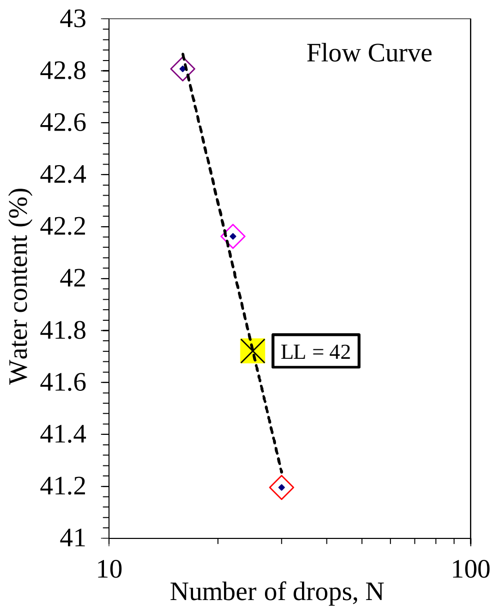
Plastic Limit

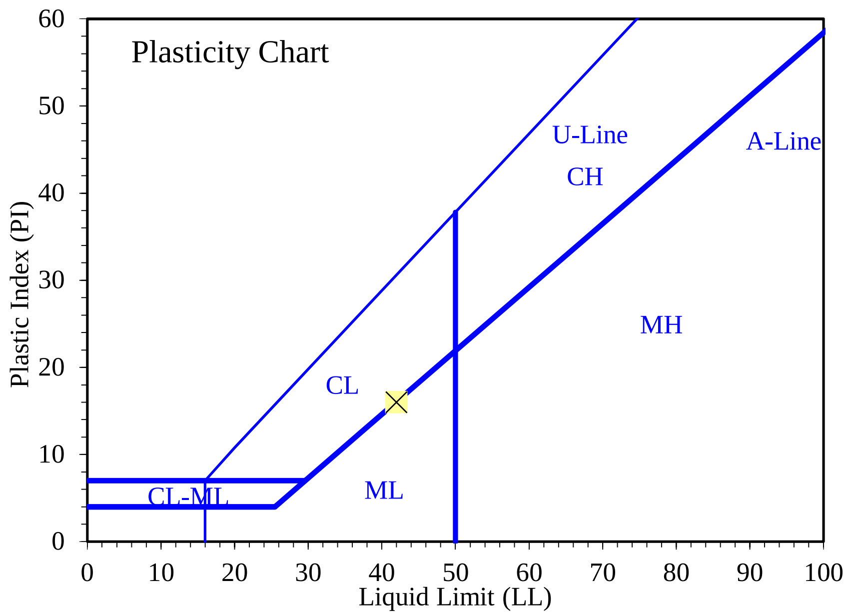
Determination No	1	2				
Wet Soil + Tare (g)	27.80	29.30				
Dry Soil + Tare (g)	26.51	27.85				
Water Loss (g)	1.29	1.45				
Tare (g)	21.54	22.29				
Dry Soil (g)	4.97	5.56				
Water Content, w (%)	25.96	26.08				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	30	22	16			
Wet Soil + Tare (g)	30.75	31.15	30.18			
Dry Soil + Tare (g)	28.27	28.46	27.71			
Water Loss (g)	2.48	2.69	2.47			
Tare (g)	22.25	22.08	21.94			
Dry Soil (g)	6.02	6.38	5.77			
Water Content, w (%)	41.20	42.16	42.81			
One-Point LL (%)	42	42				

Liquid Limit, LL (%)	**42**
Plastic Limit, PL (%)	**26**
Plasticity Index, PI (%)	**16**





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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: **Summit/Horizon Neighborhood**
No: **01628-013**
Location: Eden, UT
Date: 7/19/2016
By: BRR

Boring No.: **TP-7**
Sample:
Depth: **2.0'**
Description: Reddish brown lean clay

Preparation method: Wet
Liquid limit test method: Multipoint

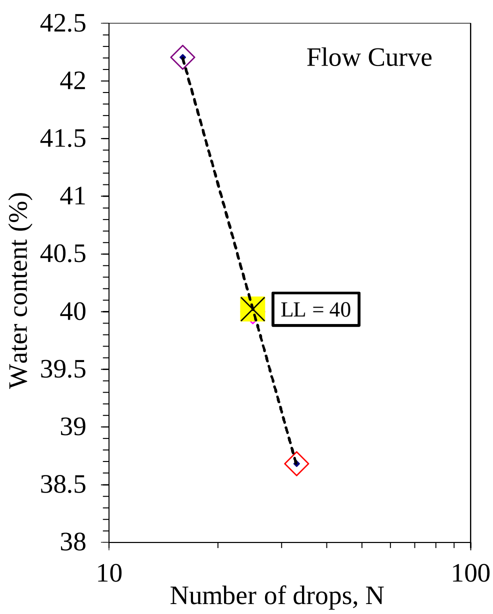
Plastic Limit

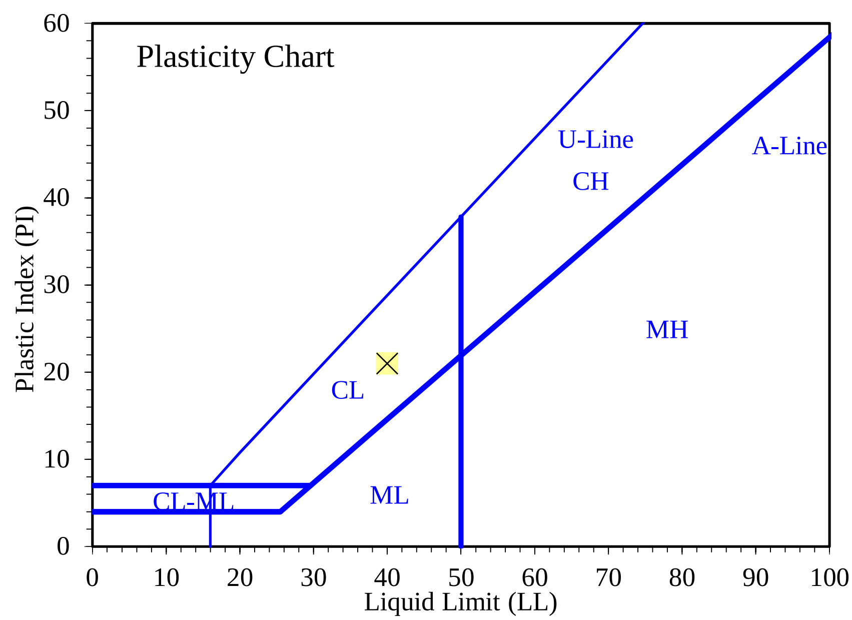
Determination No	1	2				
Wet Soil + Tare (g)	27.95	28.39				
Dry Soil + Tare (g)	26.99	27.38				
Water Loss (g)	0.96	1.01				
Tare (g)	21.92	22.02				
Dry Soil (g)	5.07	5.36				
Water Content, w (%)	18.93	18.84				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	33	25	16			
Wet Soil + Tare (g)	31.63	28.75	29.41			
Dry Soil + Tare (g)	28.93	26.75	27.19			
Water Loss (g)	2.70	2.00	2.22			
Tare (g)	21.95	21.75	21.93			
Dry Soil (g)	6.98	5.00	5.26			
Water Content, w (%)	38.68	40.00	42.21			
One-Point LL (%)		40				

Liquid Limit, LL (%)	**40**
Plastic Limit, PL (%)	**19**
Plasticity Index, PI (%)	**21**





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Liquid Limit, Plastic Limit, and Plasticity Index of Soils

(ASTM D4318)

IGES

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BRR

Boring No.: TP-8
Sample:
Depth: 8.0'
Description: Light brown silt

Preparation method: Wet
Liquid limit test method: Multipoint

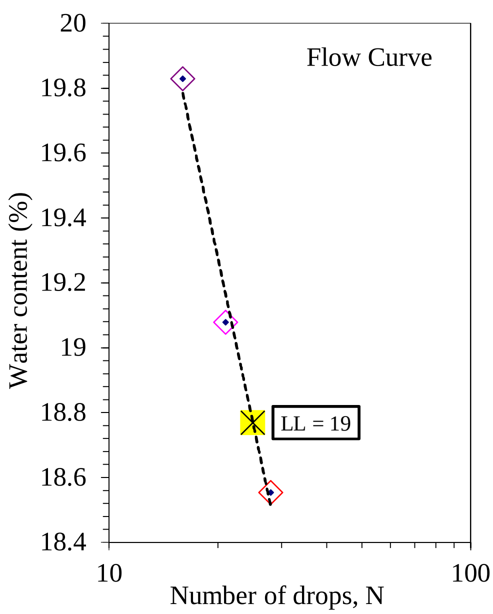
Plastic Limit

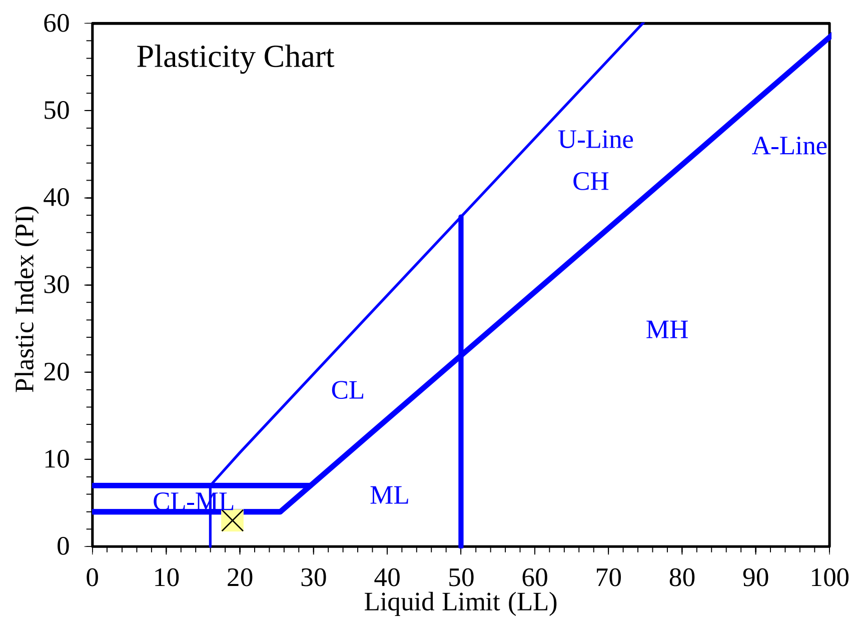
Determination No	1	2				
Wet Soil + Tare (g)	29.69	32.08				
Dry Soil + Tare (g)	28.60	30.70				
Water Loss (g)	1.09	1.38				
Tare (g)	21.95	22.11				
Dry Soil (g)	6.65	8.59				
Water Content, w (%)	16.39	16.07				

Liquid Limit

Determination No	1	2	3			
Number of Drops, N	28	21	16			
Wet Soil + Tare (g)	33.30	31.53	31.87			
Dry Soil + Tare (g)	31.53	30.04	30.24			
Water Loss (g)	1.77	1.49	1.63			
Tare (g)	21.99	22.23	22.02			
Dry Soil (g)	9.54	7.81	8.22			
Water Content, w (%)	18.55	19.08	19.83			
One-Point LL (%)	19	19				

Liquid Limit, LL (%)	**19**
Plastic Limit, PL (%)	**16**
Plasticity Index, PI (%)	**3**





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Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

IGES

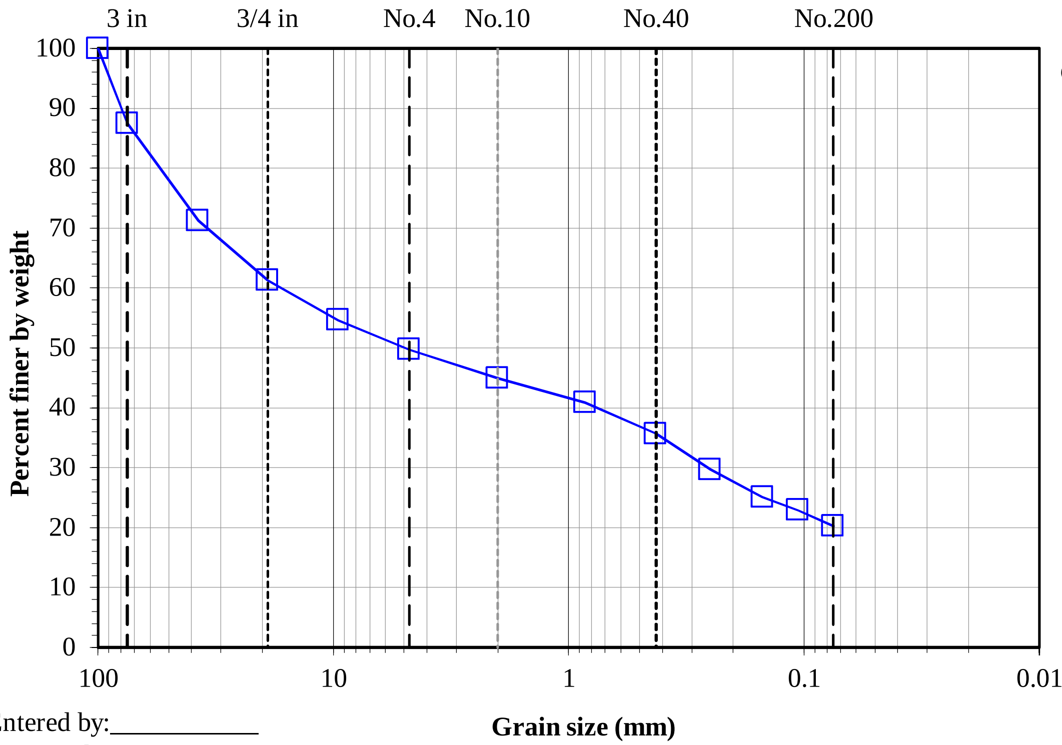
Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BSS/IM

Boring No.: TP-1
Sample:
Depth: 8.0'
Description: Reddish brown clayey gravel with sand

Split:	Yes	
Split sieve:	3/4"	
	Moist	Dry
Total sample wt. (g):	26939.70	26161.31
+3/4" Coarse fraction (g):	10192.60	10122.11
-3/4" Split fraction (g):	2008.05	1923.17
Split fraction:	0.613	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	5698.50	2416.73
Dry soil + tare (g):	5662.80	2331.85
Tare (g):	536.60	408.68
Water content (%):	0.7	4.4

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer
8"	-	200	-
6"	-	150	-
4"	-	100	100.0
3"	3274.10	75	87.5
1.5"	7527.97	37.5	71.2
3/4"	10122.10	19	61.3 ←Split
3/8"	210.17	9.5	54.6
No.4	363.96	4.75	49.7
No.10	514.57	2	44.9
No.20	642.32	0.85	40.8
No.40	804.47	0.425	35.7
No.60	992.49	0.25	29.7
No.100	1136.58	0.15	25.1
No.140	1205.69	0.106	22.9
No.200	1287.23	0.075	20.3



Gravel (%): 50.3
Sand (%): 29.4
Fines (%): 20.3

Entered by:___________
Reviewed:___________

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

IGES

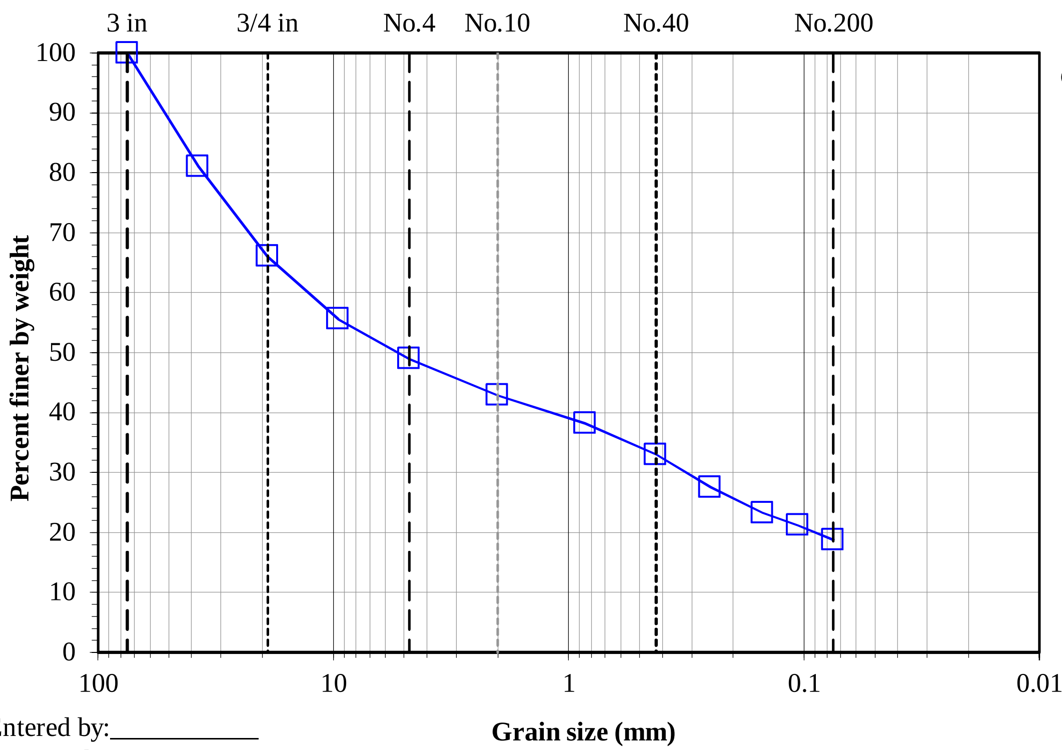
Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BSS/IM

Boring No.: TP-3
Sample:
Depth: 7.0'
Description: Reddish brown silty, clayey gravel with sand

Split:	Yes	
Split sieve:	3/4"	
	Moist	Dry
Total sample wt. (g):	4562.64	4317.59
+3/4" Coarse fraction (g):	1499.35	1466.17
-3/4" Split fraction (g):	1595.03	1484.71
Split fraction:	0.660	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	2504.16	1911.57
Dry soil + tare (g):	2464.49	1801.25
Tare (g):	711.53	316.54
Water content (%):	2.3	7.4

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer
8"	-	200	-
6"	-	150	-
4"	-	100	-
3"	-	75	100.0
1.5"	818.08	37.5	81.1
3/4"	1466.17	19	66.0 ←Split
3/8"	235.32	9.5	55.6
No.4	383.61	4.75	49.0
No.10	520.45	2	42.9
No.20	626.71	0.85	38.2
No.40	743.17	0.425	33.0
No.60	865.96	0.25	27.5
No.100	962.05	0.15	23.2
No.140	1008.41	0.106	21.2
No.200	1062.31	0.075	18.8



Gravel (%): 51.0
Sand (%): 30.2
Fines (%): 18.8

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Reviewed:___________

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Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

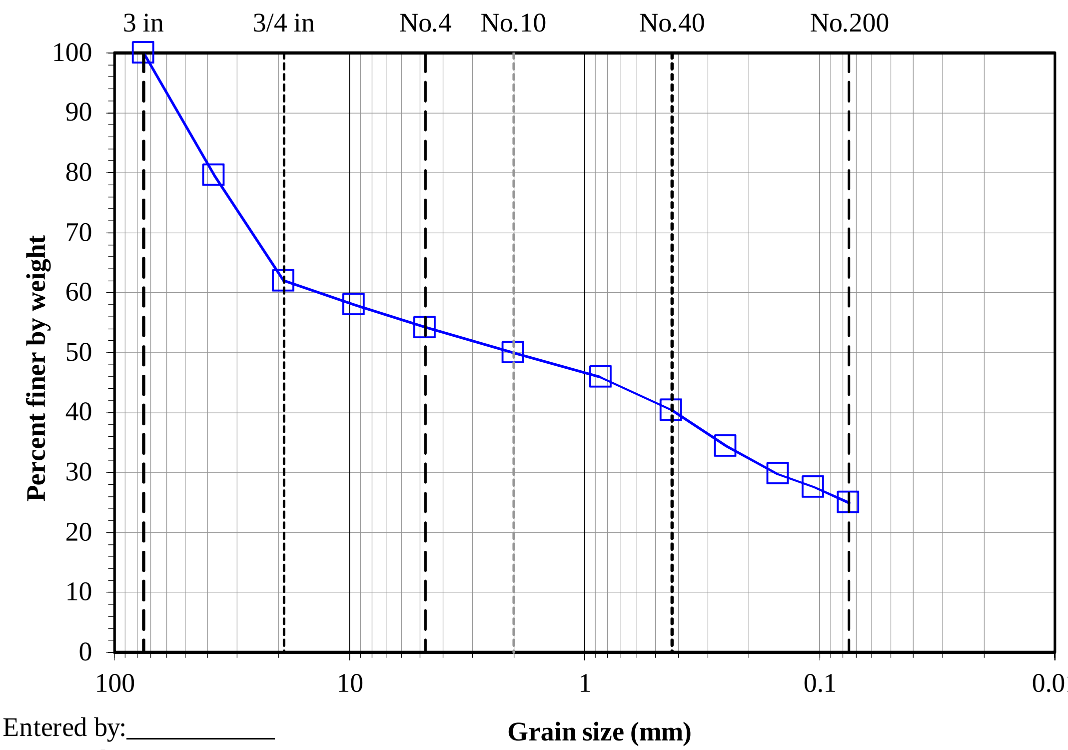
Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: BSS/IM

Boring No.: TP-4
Sample:
Depth: 5.0'
Description: Red clayey gravel with sand

Split:	Yes	
Split sieve:	3/4"	
	Moist	Dry
Total sample wt. (g):	4471.61	4216.82
+3/4" Coarse fraction (g):	1656.24	1604.80
-3/4" Split fraction (g):	1496.05	1387.99
Split fraction:	0.619	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	2425.67	1827.52
Dry soil + tare (g):	2363.01	1719.46
Tare (g):	408.18	331.47
Water content (%):	3.2	7.8

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	100.0	
1.5"	863.70	37.5	79.5	
3/4"	1604.80	19	61.9	←Split
3/8"	89.36	9.5	58.0	
No.4	174.96	4.75	54.1	
No.10	269.41	2	49.9	
No.20	360.69	0.85	45.8	
No.40	483.46	0.425	40.4	
No.60	617.68	0.25	34.4	
No.100	721.72	0.15	29.7	
No.140	770.57	0.106	27.6	
No.200	829.84	0.075	24.9	



Gravel (%): 45.9
Sand (%): 29.2
Fines (%): 24.9

Entered by:___________
Reviewed:___________

Z:\PROJECTS\01628_Powder_Mountain\013_Summit_Horizon\[GSDv2.xlsx]3

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

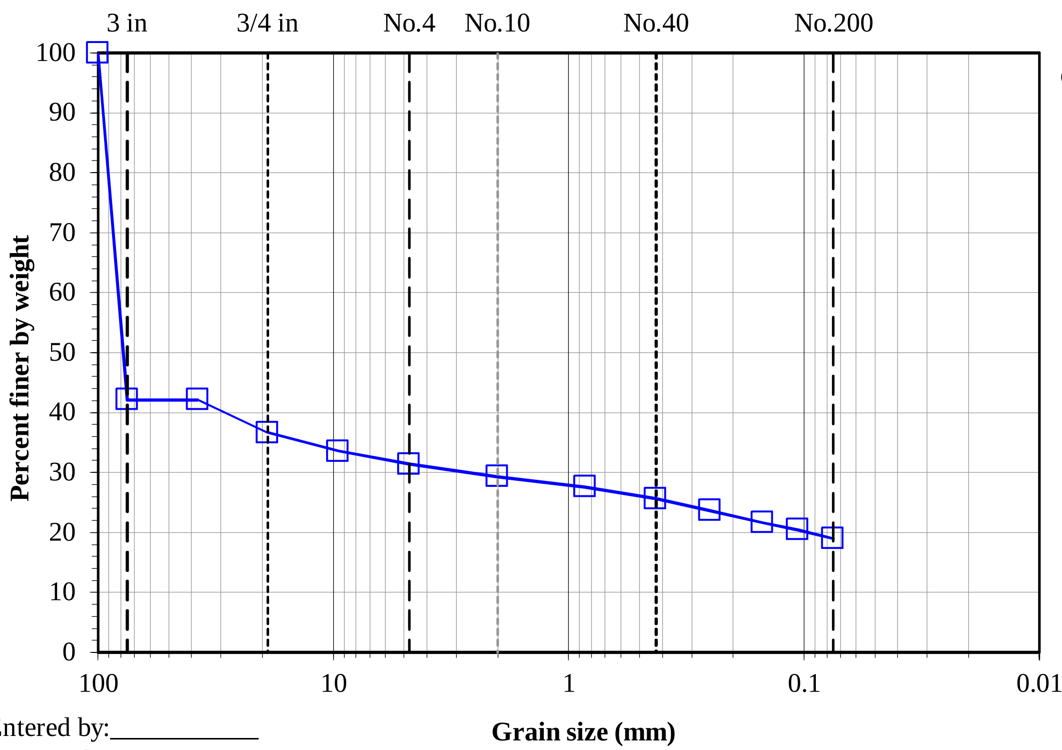
Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/18/2016
By: BSS/IM

Boring No.: TP-5
Sample:
Depth: 2.0'
Description: Dark brown clayey gravel

Split:	Yes	
Split sieve:	3/8"	
	Moist	Dry
Total sample wt. (g):	4506.59	4152.58
+3/8" Coarse fraction (g):	2808.81	2760.60
-3/8" Split fraction (g):	219.30	179.80
Split fraction:	0.335	

Water content data	C.F.(+3/8")	S.F.(-3/8")
Moist soil + tare (g):	3119.30	341.30
Dry soil + tare (g):	3071.09	301.80
Tare (g):	310.49	122.00
Water content (%):	1.7	22.0

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	100.0	
3"	2403.63	75	42.1	
1.5"	2403.63	37.5	42.1	
3/4"	2631.56	19	36.6	
3/8"	2760.60	9.5	33.5	←Split
No.4	11.33	4.75	31.4	
No.10	22.74	2	29.3	
No.20	31.76	0.85	27.6	
No.40	42.28	0.425	25.6	
No.60	53.13	0.25	23.6	
No.100	63.76	0.15	21.6	
No.140	70.08	0.106	20.5	
No.200	78.28	0.075	18.9	



Gravel (%): 68.6
Sand (%): 12.5
Fines (%): 18.9

Entered by:___________
Reviewed:___________

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

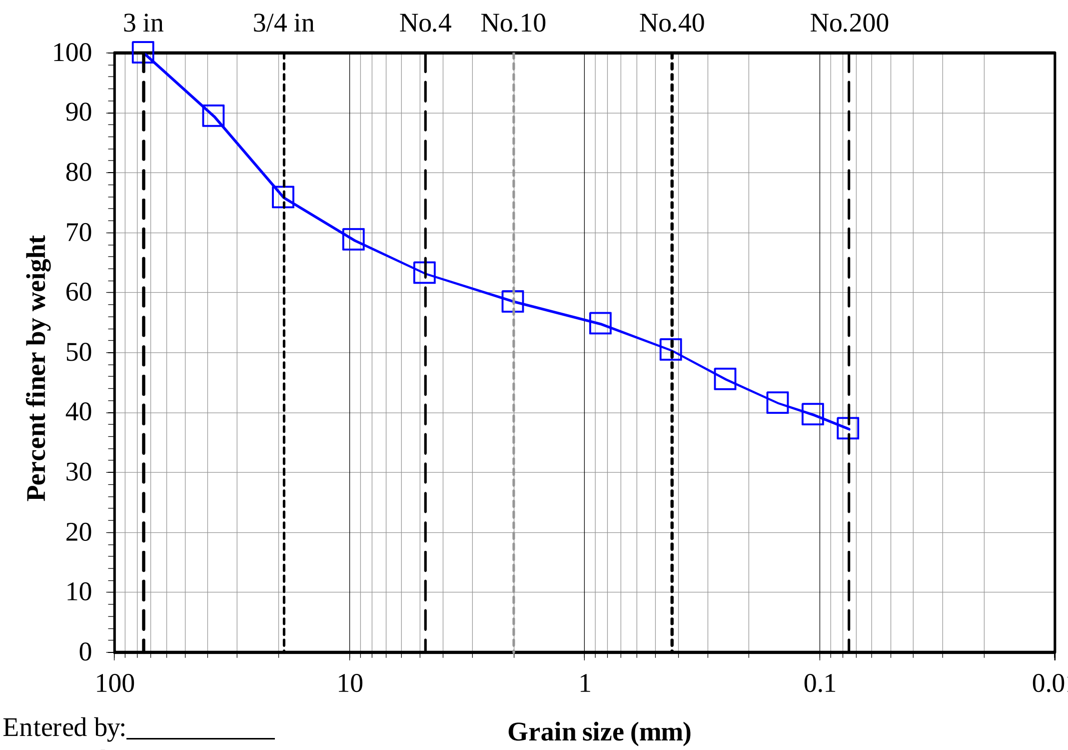
Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/18/2016
By: BSS/IM

Boring No.: TP-7
Sample:
Depth: 2.0'
Description: Reddish brown clayey gravel with sand

	Moist	Dry
Split:	Yes	
Split sieve:	3/4"	
Total sample wt. (g):	3728.40	3330.50
+3/4" Coarse fraction (g):	819.59	805.13
-3/4" Split fraction (g):	1404.12	1219.03
Split fraction:	0.758	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	1130.55	1737.24
Dry soil + tare (g):	1116.09	1552.15
Tare (g):	310.96	333.12
Water content (%):	1.8	15.2

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer	
8"	-	200	-	
6"	-	150	-	
4"	-	100	-	
3"	-	75	100.0	
1.5"	355.52	37.5	89.3	
3/4"	805.13	19	75.8	←Split
3/8"	114.33	9.5	68.7	
No.4	203.50	4.75	63.2	
No.10	279.94	2	58.4	
No.20	339.33	0.85	54.7	
No.40	409.88	0.425	50.3	
No.60	487.68	0.25	45.5	
No.100	550.97	0.15	41.6	
No.140	583.00	0.106	39.6	
No.200	620.89	0.075	37.2	



Gravel (%): 36.8
Sand (%): 26.0
Fines (%): 37.2

Entered by:___________
Reviewed:___________

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

(ASTM D6913)

IGES

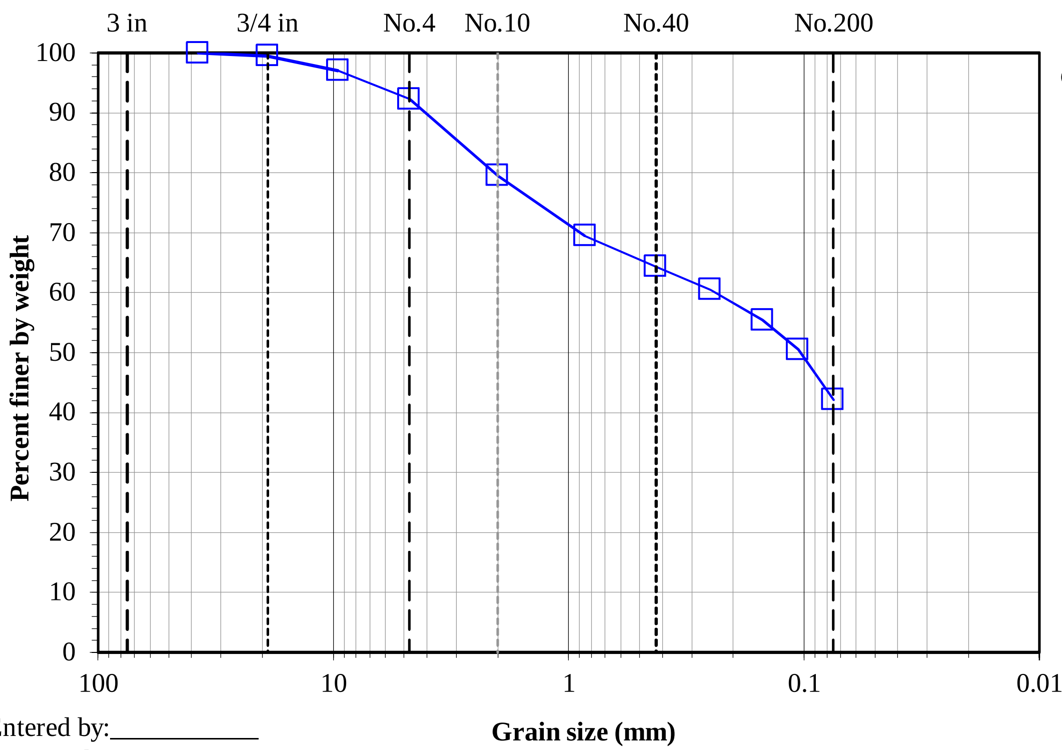
Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/18/2016
By: BSS/IM

Boring No.: TP-8
Sample:
Depth: 8.0'
Description: Yellowish brown silty sand

	Moist	Dry
Split:	Yes	
Split sieve:	3/4"	
Total sample wt. (g):	3454.16	2987.27
+3/4" Coarse fraction (g):	18.75	15.41
-3/4" Split fraction (g):	1435.44	1241.75
Split fraction:	0.995	

Water content data	C.F.(+3/4")	S.F.(-3/4")
Moist soil + tare (g):	140.62	1844.47
Dry soil + tare (g):	137.28	1650.78
Tare (g):	121.87	409.03
Water content (%):	21.7	15.6

Sieve	Accum. Wt. Ret. (g)	Grain Size (mm)	Percent Finer
8"	-	200	-
6"	-	150	-
4"	-	100	-
3"	-	75	-
1.5"	-	37.5	100.0
3/4"	15.41	19	99.5 ←Split
3/8"	30.77	9.5	97.0
No.4	89.98	4.75	92.3
No.10	249.37	2	79.5
No.20	374.59	0.85	69.5
No.40	438.37	0.425	64.4
No.60	486.41	0.25	60.5
No.100	549.84	0.15	55.4
No.140	611.62	0.106	50.5
No.200	715.73	0.075	42.1



Gravel (%): 7.7
Sand (%): 50.1
Fines (%): 42.1

Entered by:___________
Reviewed:___________

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Amount of Material in Soil Finer than the No. 200 (75µm) Sieve

(ASTM D1140)

IGES

Project: **Summit/Horizon Neighborhood**
No: **01628-0013**
Location: Eden, UT
Date: 7/18/2016
By: NB/IM

Sample Info.	Boring No.	TP-1	TP-2						
	Sample		2						
	Depth	13.0'	5.0'						
	Split	No	No						
	Split Sieve*								
	Method	B	B						
	Specimen soak time (min)	390	420						
	Moist total sample wt. (g)	137.40	276.80						
	Moist coarse fraction (g)								
	Moist split fraction + tare (g)								
	Split fraction tare (g)								
	Dry split fraction (g)								
	Dry retained No. 200 + tare (g)	125.27	163.39						
	Wash tare (g)	120.72	124.48						
	No. 200 Dry wt. retained (g)	4.55	38.91						
	Split sieve* Dry wt. retained (g)								
	Dry total sample wt. (g)	106.65	219.55						
Coarse Fraction	Moist soil + tare (g)								
	Dry soil + tare (g)								
	Tare (g)								
	Water content (%)								
Split Fraction	Moist soil + tare (g)	258.12	401.28						
	Dry soil + tare (g)	227.37	344.03						
	Tare (g)	120.72	124.48						
	Water content (%)	28.83	26.08						
	Percent passing split sieve* (%)								
	Percent passing No. 200 sieve (%)	**95.7**	**82.3**						

Entered by:___________

Reviewed:___________

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Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/18/2016
By: NB

Boring No.: TP-2
Sample: 2
Depth: 5.0'
Sample Description: Reddish brown clay with gravel
Sample type: Undisturbed-trimmed from thin-wall

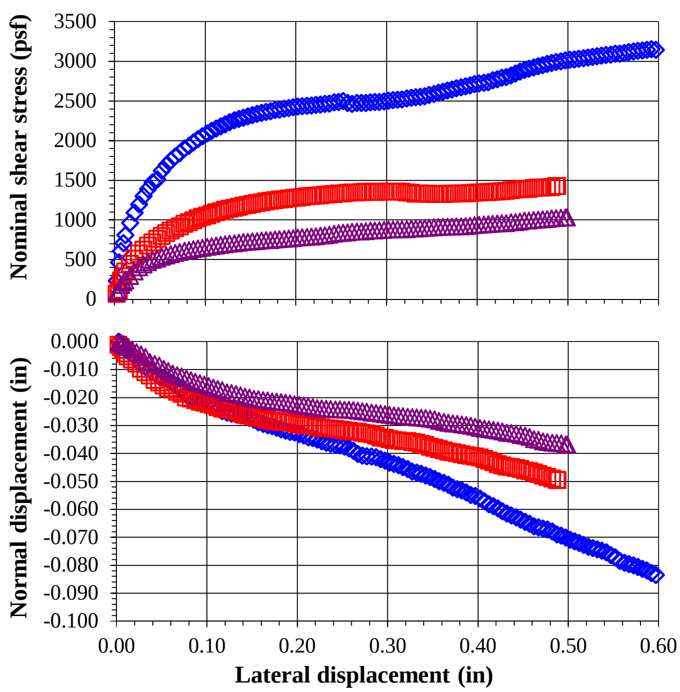
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0005
Specific gravity, Gs: 2.70 Assumed

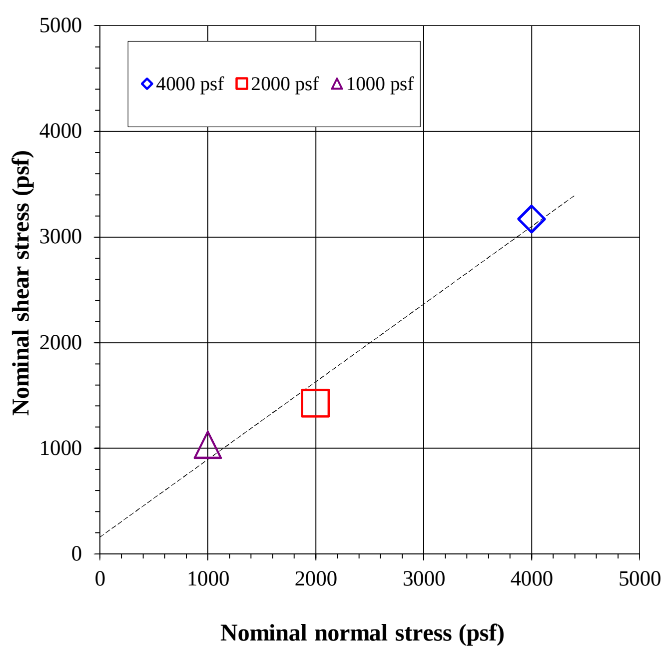
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4000		2000		1000	
Peak shear stress (psf)	3169		1425		1032	
Lateral displacement at peak (in)	0.622		0.488		0.499	
Load Duration (min)	851		861		873	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	1.0000	0.9288	1.0000	0.9284	1.0000	0.9715
Sample diameter (in)	2.416	2.416	2.416	2.416	2.416	2.416
Wt. rings + wet soil (g)	177.48	179.48	178.28	180.29	182.24	186.62
Wt. rings (g)	42.08	42.08	42.98	42.98	45.20	45.20
Wet soil + tare (g)	401.28		401.28		401.28	
Dry soil + tare (g)	344.03		344.03		344.03	
Tare (g)	124.48		124.48		124.48	
Water content (%)	26.1	27.9	26.1	28.0	26.1	30.1
Dry unit weight (pcf)	89.2	96.0	89.2	96.0	90.3	92.9
Void ratio, e, for assumed Gs	0.89	0.75	0.89	0.75	0.87	0.81
Saturation (%)*	79.2	100.0	79.1	100.0	81.3	100.0

ϕ' (deg)	36
c' (psf)	160

Average of 3 samples	Initial	Pre-shear
Water content (%)	26.1	28.7
Dry unit weight (pcf)	89.6	95.0

*Pre-shear saturation set to 100% for phase calculations





Comments:
Test specimens were sheared to the maximum available horizontal displacement.

Entered by:___________
Reviewed:___________

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Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Summit/Horizon Neighborhood **Boring No.:** TP-2
No: 01628-013 **Sample:** 2
Location: Eden, UT **Depth:** 5.0'

Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	232	0.000	0.002	67	-0.001	0.002	84	0.000
0.005	464	-0.001	0.005	119	-0.002	0.005	100	0.000
0.007	608	-0.002	0.007	235	-0.003	0.007	179	-0.001
0.010	709	-0.003	0.010	287	-0.004	0.010	209	-0.001
0.012	807	-0.003	0.012	347	-0.005	0.012	236	-0.002
0.017	961	-0.004	0.017	443	-0.006	0.017	292	-0.003
0.022	1090	-0.006	0.022	512	-0.008	0.022	348	-0.004
0.027	1196	-0.008	0.027	573	-0.010	0.027	401	-0.005
0.032	1301	-0.009	0.032	624	-0.011	0.032	437	-0.006
0.037	1391	-0.010	0.037	673	-0.012	0.037	466	-0.007
0.042	1474	-0.011	0.042	716	-0.014	0.042	493	-0.008
0.047	1512	-0.013	0.047	758	-0.015	0.047	511	-0.009
0.052	1621	-0.013	0.052	797	-0.016	0.052	534	-0.010
0.057	1688	-0.014	0.057	834	-0.017	0.057	549	-0.010
0.062	1747	-0.015	0.062	863	-0.017	0.062	568	-0.011
0.067	1801	-0.016	0.067	891	-0.018	0.067	581	-0.012
0.072	1842	-0.017	0.072	917	-0.019	0.072	596	-0.013
0.077	1902	-0.019	0.077	948	-0.020	0.077	610	-0.013
0.082	1935	-0.020	0.082	971	-0.020	0.082	618	-0.014
0.087	1979	-0.021	0.087	994	-0.021	0.087	629	-0.014
0.092	2020	-0.021	0.092	1019	-0.022	0.092	639	-0.015
0.097	2054	-0.021	0.097	1038	-0.022	0.097	648	-0.015
0.102	2092	-0.022	0.102	1055	-0.022	0.102	658	-0.016
0.107	2123	-0.023	0.107	1074	-0.023	0.107	668	-0.016
0.112	2154	-0.024	0.112	1093	-0.024	0.112	676	-0.017
0.117	2182	-0.025	0.117	1106	-0.024	0.117	685	-0.017
0.122	2203	-0.025	0.122	1126	-0.024	0.122	693	-0.018
0.127	2234	-0.026	0.127	1140	-0.024	0.127	699	-0.018
0.132	2257	-0.026	0.132	1151	-0.025	0.132	704	-0.019
0.137	2275	-0.026	0.137	1164	-0.026	0.137	711	-0.019
0.142	2291	-0.027	0.142	1178	-0.026	0.142	718	-0.020
0.147	2306	-0.027	0.147	1186	-0.026	0.147	724	-0.020
0.152	2322	-0.028	0.152	1200	-0.027	0.152	727	-0.021
0.157	2337	-0.029	0.157	1210	-0.027	0.157	732	-0.021
0.162	2350	-0.029	0.162	1220	-0.028	0.162	739	-0.021
0.167	2360	-0.030	0.167	1230	-0.028	0.167	744	-0.021
0.172	2371	-0.030	0.172	1238	-0.028	0.172	745	-0.021
0.177	2386	-0.031	0.177	1250	-0.029	0.177	751	-0.021
0.182	2391	-0.031	0.182	1255	-0.029	0.182	756	-0.022
0.187	2401	-0.032	0.187	1261	-0.029	0.187	761	-0.022
0.192	2412	-0.032	0.192	1270	-0.029	0.192	763	-0.022
0.197	2422	-0.033	0.197	1276	-0.029	0.197	769	-0.022
0.202	2432	-0.033	0.202	1281	-0.030	0.202	776	-0.023
0.207	2435	-0.034	0.207	1290	-0.030	0.207	780	-0.023
0.212	2438	-0.034	0.212	1295	-0.030	0.212	783	-0.023
0.217	2443	-0.034	0.217	1300	-0.030	0.217	789	-0.023
0.222	2450	-0.035	0.222	1303	-0.031	0.222	792	-0.024
0.227	2453	-0.036	0.227	1310	-0.031	0.227	799	-0.024
0.232	2463	-0.036	0.232	1316	-0.031	0.232	805	-0.024
0.237	2466	-0.037	0.237	1321	-0.031	0.237	813	-0.024
0.242	2481	-0.037	0.242	1324	-0.031	0.242	825	-0.024
0.247	2486	-0.037	0.247	1331	-0.032	0.247	831	-0.024
0.252	2499	-0.038	0.252	1332	-0.032	0.252	837	-0.024
0.257	2474	-0.038	0.257	1341	-0.032	0.257	842	-0.024
0.262	2463	-0.039	0.262	1345	-0.032	0.262	847	-0.025
0.267	2476	-0.040	0.267	1346	-0.032	0.267	853	-0.025
0.272	2476	-0.041	0.272	1348	-0.033	0.272	854	-0.025
0.277	2476	-0.041	0.277	1354	-0.033	0.277	858	-0.025
0.282	2484	-0.041	0.282	1355	-0.033	0.282	864	-0.025
0.287	2486	-0.041	0.287	1355	-0.034	0.287	866	-0.026
0.292	2486	-0.042	0.292	1355	-0.034	0.292	869	-0.026
0.297	2494	-0.043	0.297	1355	-0.035	0.297	873	-0.026
0.302	2502	-0.043	0.302	1358	-0.035	0.302	878	-0.026
0.307	2510	-0.044	0.307	1359	-0.035	0.307	879	-0.026
0.312	2515	-0.044	0.312	1357	-0.035	0.312	883	-0.027

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

IGES

Project: Summit/Horizon Neighborhood **Boring No.:** TP-2

No: 01628-013 **Sample:** 2

Location: Eden, UT **Depth:** 5.0'

Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.317	2520	-0.045	0.317	1353	-0.036	0.317	880	-0.027
0.322	2530	-0.046	0.322	1343	-0.036	0.322	881	-0.027
0.327	2538	-0.046	0.327	1340	-0.036	0.327	888	-0.027
0.332	2546	-0.047	0.332	1330	-0.036	0.332	889	-0.027
0.337	2551	-0.047	0.337	1331	-0.037	0.337	895	-0.027
0.342	2561	-0.048	0.342	1327	-0.037	0.342	898	-0.027
0.347	2574	-0.048	0.347	1330	-0.038	0.347	900	-0.028
0.352	2587	-0.049	0.352	1325	-0.038	0.352	905	-0.028
0.357	2602	-0.050	0.357	1325	-0.038	0.357	909	-0.028
0.362	2618	-0.051	0.362	1325	-0.039	0.362	911	-0.029
0.367	2631	-0.051	0.367	1328	-0.039	0.367	913	-0.029
0.372	2644	-0.052	0.372	1328	-0.040	0.372	915	-0.029
0.377	2662	-0.053	0.377	1331	-0.040	0.377	916	-0.029
0.382	2672	-0.053	0.382	1331	-0.040	0.382	916	-0.030
0.387	2685	-0.054	0.387	1334	-0.041	0.387	922	-0.030
0.392	2695	-0.055	0.392	1336	-0.041	0.392	925	-0.030
0.397	2711	-0.055	0.397	1341	-0.041	0.397	932	-0.031
0.402	2721	-0.056	0.402	1345	-0.041	0.402	938	-0.031
0.407	2731	-0.057	0.407	1350	-0.042	0.407	942	-0.031
0.412	2739	-0.058	0.412	1350	-0.043	0.412	945	-0.031
0.417	2760	-0.059	0.417	1355	-0.043	0.417	950	-0.032
0.422	2775	-0.060	0.422	1359	-0.044	0.422	953	-0.032
0.427	2793	-0.061	0.427	1361	-0.044	0.427	958	-0.032
0.432	2803	-0.061	0.432	1367	-0.045	0.432	957	-0.033
0.437	2821	-0.062	0.437	1372	-0.045	0.437	968	-0.033
0.442	2847	-0.063	0.442	1381	-0.045	0.442	972	-0.033
0.447	2870	-0.064	0.447	1385	-0.046	0.447	978	-0.033
0.452	2891	-0.064	0.452	1388	-0.046	0.452	988	-0.034
0.457	2906	-0.065	0.457	1390	-0.046	0.457	992	-0.034
0.462	2922	-0.066	0.462	1400	-0.047	0.462	998	-0.035
0.467	2937	-0.066	0.467	1404	-0.047	0.467	1002	-0.035
0.472	2950	-0.067	0.472	1406	-0.048	0.472	1005	-0.036
0.477	2966	-0.067	0.477	1410	-0.048	0.477	1010	-0.036
0.482	2979	-0.068	0.482	1415	-0.049	0.482	1017	-0.036
0.487	2991	-0.069	0.487	1424	-0.049	0.487	1023	-0.036
0.492	2999	-0.069	0.488	1425	-0.050	0.492	1027	-0.036
0.497	3012	-0.070				0.497	1029	-0.037
0.502	3020	-0.071				0.499	1032	-0.037
0.507	3022	-0.071						
0.512	3030	-0.072						
0.517	3040	-0.073						
0.522	3046	-0.073						
0.527	3056	-0.074						
0.532	3066	-0.074						
0.537	3074	-0.075						
0.542	3082	-0.075						
0.547	3084	-0.076						
0.552	3095	-0.077						
0.557	3097	-0.078						
0.562	3105	-0.079						
0.567	3118	-0.080						
0.572	3123	-0.080						
0.577	3131	-0.081						
0.582	3138	-0.081						
0.587	3144	-0.082						
0.592	3151	-0.083						
0.597	3146	-0.084						
0.602	3151	-0.084						
0.607	3156	-0.085						
0.612	3162	-0.086						
0.617	3164	-0.087						
0.622	3169	-0.087						

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

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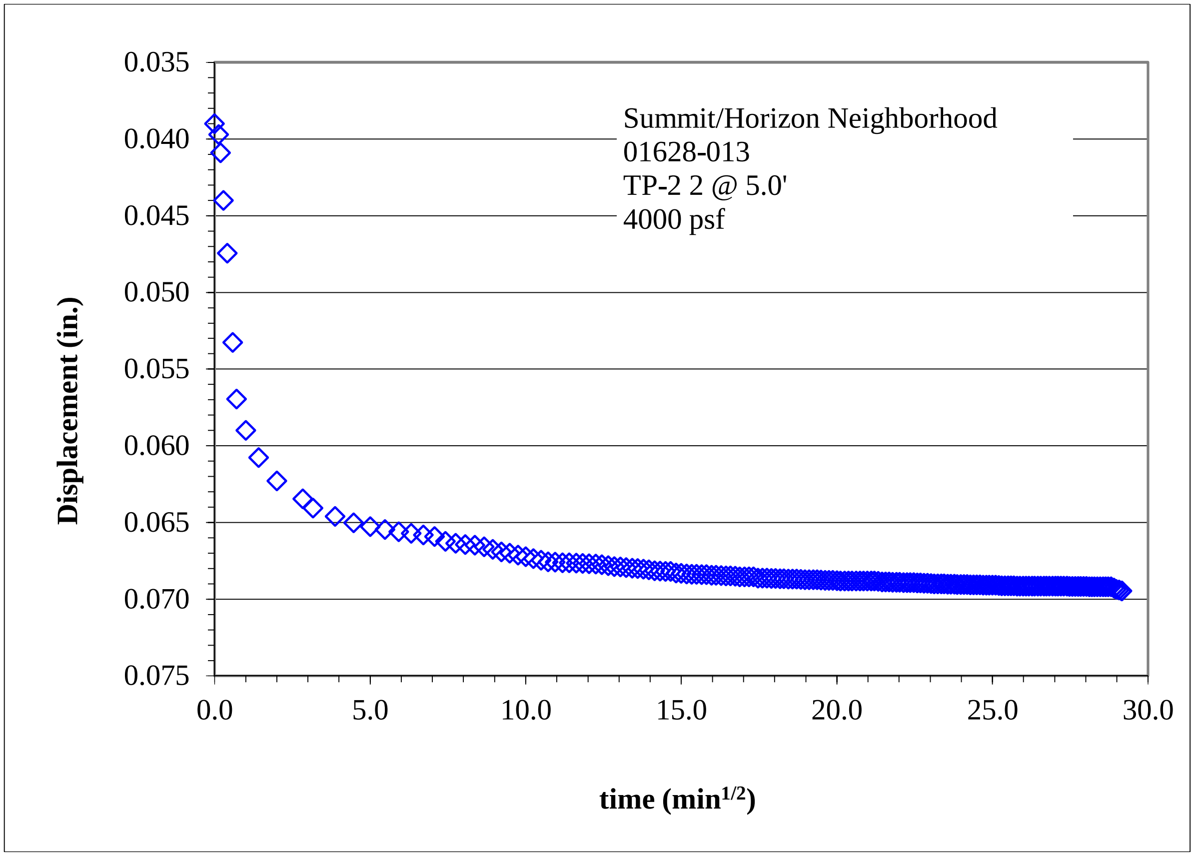
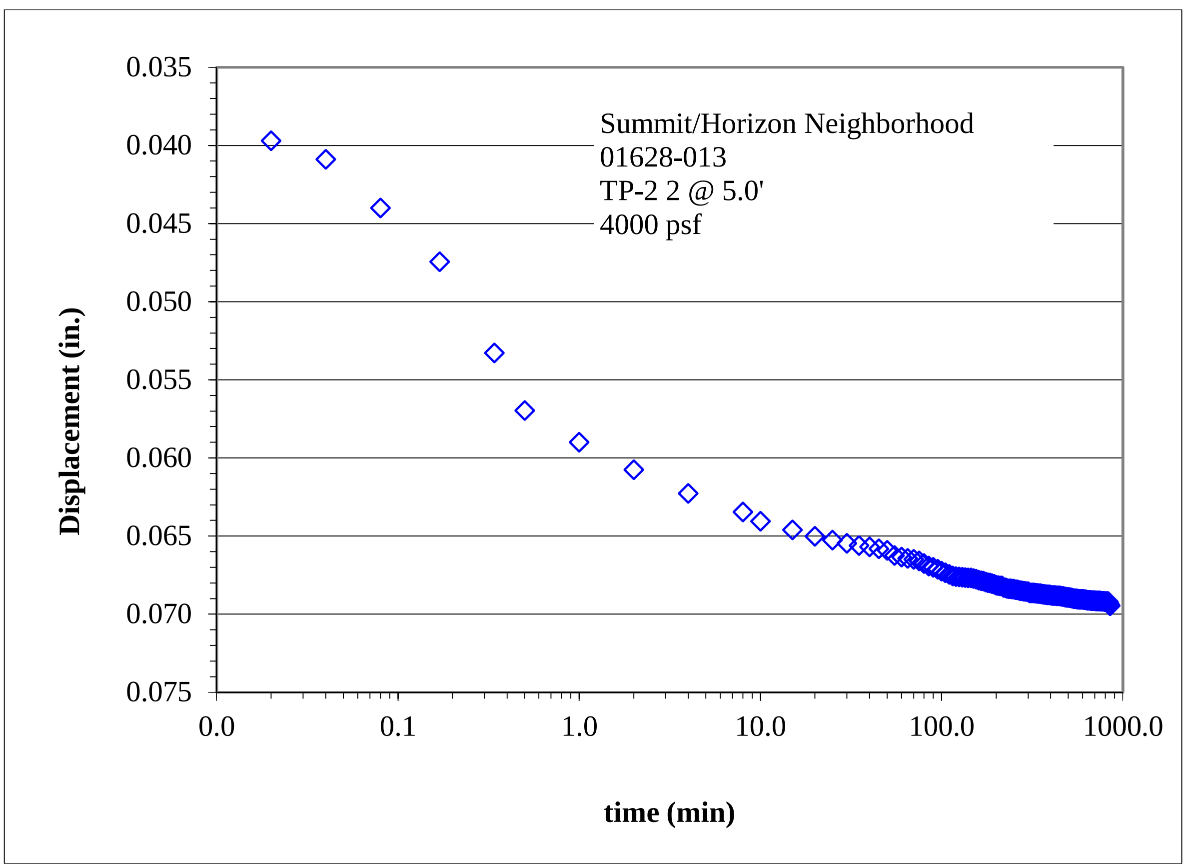
Project: **Summit/Horizon Neighborhood** **Boring No.:** **TP-2**
No: **01628-013** **Sample:** **2**
Location: Eden, UT **Depth:** **5.0'**





Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT
Date: 7/19/2016
By: JDF

Boring No.: TP-12
Sample:
Depth: 6.0'
Sample Description: Reddish brown clay with sand
Sample type: Undisturbed-trimmed from thin-wall

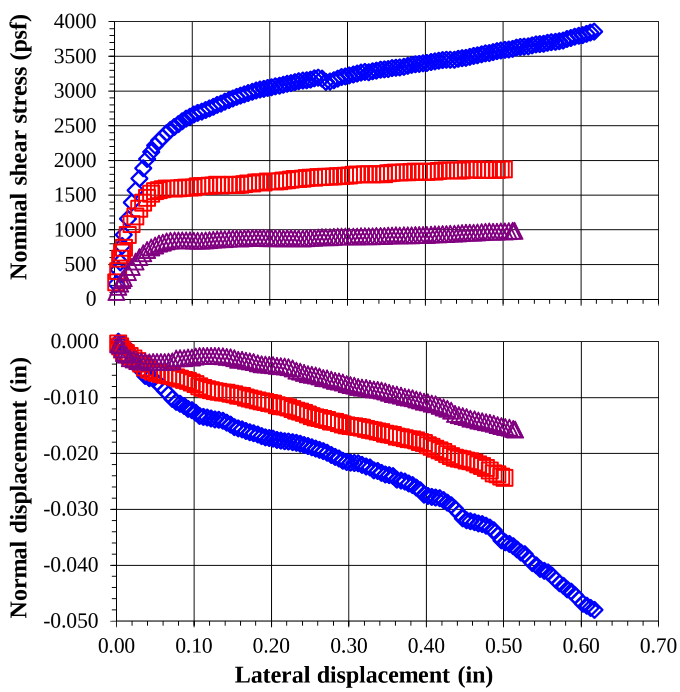
Test type: Inundated
Lateral displacement (in.): 0.3
Shear rate (in./min): 0.0009
Specific gravity, Gs: 2.70 Assumed

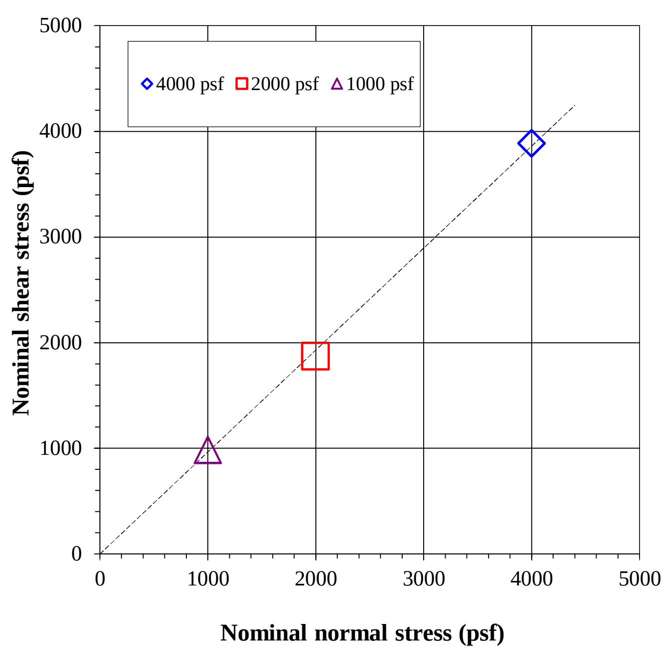
	Sample 1		Sample 2		Sample 3	
Nominal normal stress (psf)	4000		2000		1000	
Peak shear stress (psf)	3886		1871		983	
Lateral displacement at peak (in)	0.634		0.501		0.512	
Load Duration (min)	130		156		174	
	Initial	Pre-shear	Initial	Pre-shear	Initial	Pre-shear
Sample height (in)	1.0000	0.9626	1.0000	0.9710	1.0000	0.9821
Sample diameter (in)	2.416	2.416	2.416	2.416	2.416	2.416
Wt. rings + wet soil (g)	187.36	189.61	184.50	187.61	189.32	192.20
Wt. rings (g)	44.88	44.88	42.47	42.47	45.13	45.13
Wet soil + tare (g)	435.27		435.27		435.27	
Dry soil + tare (g)	375.12		375.12		375.12	
Tare (g)	123.23		123.23		123.23	
Water content (%)	23.9	25.8	23.9	26.6	23.9	26.4
Dry unit weight (pcf)	95.6	99.2	95.3	98.1	96.7	98.4
Void ratio, e, for assumed Gs	0.76	0.70	0.77	0.72	0.74	0.71
Saturation (%)*	84.4	100.0	83.8	100.0	86.8	100.0

ϕ' (deg)	44
c' (psf)	0

Average of 3 samples	Initial	Pre-shear
Water content (%)	23.9	26.3
Dry unit weight (pcf)	95.9	98.6

*Pre-shear saturation set to 100% for phase calculations





Comments:
Test specimens were sheared to the maximum available horizontal displacement.

Entered by:___________
Reviewed:___________

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT

Boring No.: TP-12
Sample:
Depth: 6.0'

Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.002	216	0.000	0.002	239	0.000	0.002	104	-0.001
0.005	428	0.000	0.005	402	-0.001	0.005	176	-0.001
0.007	533	0.000	0.007	582	-0.001	0.007	224	-0.001
0.010	768	-0.001	0.010	668	-0.002	0.010	273	-0.002
0.012	928	-0.001	0.012	741	-0.002	0.012	303	-0.002
0.017	1157	-0.002	0.017	925	-0.003	0.017	389	-0.003
0.022	1394	-0.003	0.022	1080	-0.003	0.022	461	-0.003
0.027	1569	-0.004	0.027	1190	-0.003	0.027	543	-0.004
0.032	1739	-0.005	0.032	1300	-0.004	0.032	606	-0.004
0.037	1886	-0.006	0.037	1390	-0.005	0.037	663	-0.004
0.042	2020	-0.007	0.042	1460	-0.005	0.042	708	-0.004
0.047	2123	-0.007	0.047	1520	-0.005	0.047	744	-0.004
0.052	2213	-0.007	0.052	1557	-0.006	0.052	773	-0.004
0.057	2285	-0.008	0.057	1571	-0.006	0.057	798	-0.004
0.062	2332	-0.009	0.062	1584	-0.006	0.062	815	-0.004
0.067	2396	-0.009	0.067	1592	-0.006	0.067	833	-0.004
0.072	2440	-0.010	0.072	1594	-0.006	0.072	841	-0.004
0.077	2481	-0.011	0.077	1592	-0.006	0.077	843	-0.003
0.082	2525	-0.011	0.082	1598	-0.007	0.082	845	-0.003
0.087	2566	-0.012	0.087	1602	-0.007	0.087	845	-0.003
0.092	2602	-0.012	0.092	1604	-0.007	0.092	845	-0.003
0.097	2631	-0.012	0.097	1607	-0.007	0.097	846	-0.003
0.102	2662	-0.013	0.102	1610	-0.008	0.102	842	-0.003
0.107	2685	-0.013	0.107	1623	-0.008	0.107	840	-0.003
0.112	2705	-0.013	0.112	1628	-0.008	0.112	842	-0.003
0.117	2729	-0.014	0.117	1630	-0.009	0.117	845	-0.003
0.122	2749	-0.014	0.122	1634	-0.009	0.122	848	-0.003
0.127	2772	-0.014	0.127	1640	-0.009	0.127	853	-0.003
0.132	2801	-0.014	0.132	1646	-0.009	0.132	857	-0.003
0.137	2819	-0.014	0.137	1640	-0.009	0.137	863	-0.003
0.142	2845	-0.014	0.142	1642	-0.009	0.142	865	-0.003
0.147	2868	-0.015	0.147	1645	-0.009	0.147	869	-0.003
0.152	2891	-0.015	0.152	1648	-0.009	0.152	871	-0.003
0.157	2914	-0.016	0.157	1645	-0.010	0.157	873	-0.003
0.162	2935	-0.016	0.162	1650	-0.010	0.162	875	-0.003
0.167	2953	-0.016	0.167	1655	-0.010	0.167	878	-0.003
0.172	2973	-0.016	0.172	1664	-0.010	0.172	881	-0.004
0.177	2989	-0.016	0.177	1667	-0.010	0.177	882	-0.004
0.182	3007	-0.017	0.182	1680	-0.011	0.182	884	-0.004
0.187	3020	-0.017	0.187	1686	-0.011	0.187	884	-0.004
0.192	3030	-0.017	0.192	1682	-0.011	0.192	882	-0.004
0.197	3043	-0.017	0.197	1693	-0.011	0.197	879	-0.004
0.202	3056	-0.017	0.202	1695	-0.011	0.202	879	-0.004
0.207	3066	-0.018	0.207	1696	-0.011	0.207	879	-0.004
0.212	3079	-0.018	0.212	1697	-0.012	0.212	876	-0.004
0.217	3092	-0.018	0.217	1708	-0.012	0.217	875	-0.004
0.222	3107	-0.018	0.222	1712	-0.012	0.222	876	-0.005
0.227	3115	-0.018	0.227	1723	-0.012	0.227	876	-0.005
0.232	3128	-0.018	0.232	1729	-0.012	0.232	874	-0.005
0.237	3141	-0.018	0.237	1729	-0.013	0.237	873	-0.005
0.242	3149	-0.018	0.242	1735	-0.013	0.242	874	-0.006
0.247	3154	-0.019	0.247	1744	-0.013	0.247	879	-0.006
0.252	3167	-0.019	0.252	1751	-0.013	0.252	881	-0.006
0.257	3182	-0.019	0.257	1750	-0.014	0.257	884	-0.006
0.262	3192	-0.019	0.262	1758	-0.014	0.262	886	-0.006
0.267	3180	-0.020	0.267	1759	-0.014	0.267	889	-0.007
0.272	3133	-0.020	0.272	1762	-0.014	0.272	891	-0.007
0.277	3136	-0.020	0.277	1763	-0.014	0.277	894	-0.007
0.282	3154	-0.021	0.282	1769	-0.014	0.282	897	-0.007
0.287	3177	-0.021	0.287	1774	-0.015	0.287	899	-0.007
0.292	3200	-0.021	0.292	1774	-0.015	0.292	899	-0.007
0.297	3213	-0.022	0.297	1775	-0.015	0.297	900	-0.008
0.302	3223	-0.022	0.302	1780	-0.015	0.302	899	-0.008
0.307	3239	-0.022	0.307	1789	-0.015	0.307	900	-0.008
0.312	3254	-0.022	0.312	1796	-0.015	0.312	901	-0.008

Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT

Boring No.: TP-12
Sample:
Depth: 6.0'

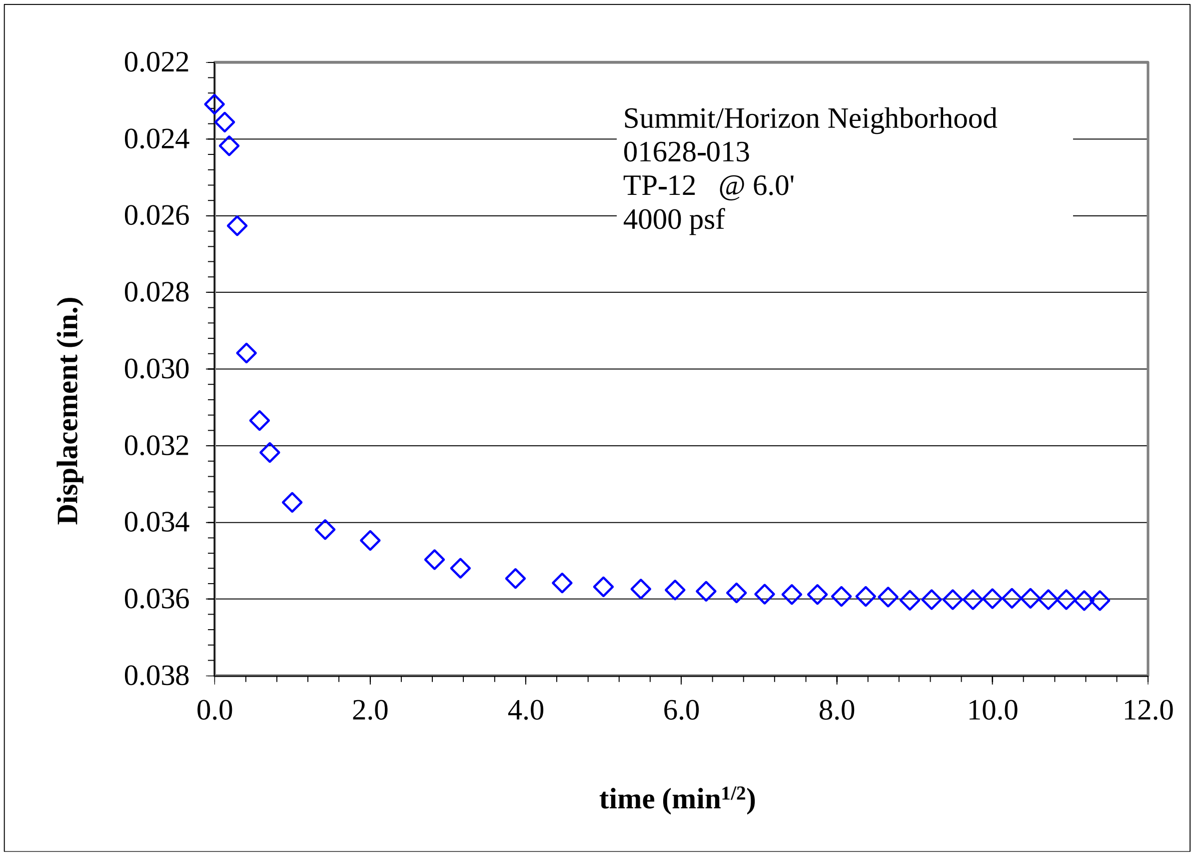
Nominal normal stress = 4000 psf			Nominal normal stress = 2000 psf			Nominal normal stress = 1000 psf		
Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)	Lateral Displacement (in.)	Nominal Shear Stress (psf)	Normal Displacement (in.)
0.317	3267	-0.022	0.317	1797	-0.015	0.317	904	-0.008
0.322	3275	-0.022	0.322	1799	-0.016	0.322	904	-0.008
0.327	3270	-0.022	0.327	1799	-0.016	0.327	905	-0.008
0.332	3285	-0.023	0.332	1797	-0.016	0.332	905	-0.009
0.337	3293	-0.023	0.337	1799	-0.016	0.337	908	-0.009
0.342	3306	-0.024	0.342	1799	-0.016	0.342	910	-0.009
0.347	3311	-0.024	0.347	1801	-0.016	0.347	912	-0.009
0.352	3319	-0.024	0.352	1813	-0.017	0.352	915	-0.009
0.357	3332	-0.024	0.357	1817	-0.017	0.357	916	-0.009
0.362	3337	-0.025	0.362	1822	-0.017	0.362	917	-0.010
0.367	3342	-0.025	0.367	1823	-0.017	0.367	916	-0.010
0.372	3350	-0.025	0.372	1826	-0.017	0.372	917	-0.010
0.377	3363	-0.025	0.377	1829	-0.017	0.377	918	-0.010
0.382	3375	-0.026	0.382	1834	-0.018	0.382	920	-0.010
0.387	3386	-0.026	0.387	1838	-0.018	0.387	923	-0.010
0.392	3393	-0.027	0.392	1838	-0.018	0.392	924	-0.011
0.397	3401	-0.027	0.397	1833	-0.018	0.397	925	-0.011
0.402	3406	-0.028	0.402	1833	-0.019	0.402	925	-0.011
0.407	3419	-0.028	0.407	1839	-0.019	0.407	929	-0.011
0.412	3427	-0.028	0.412	1839	-0.019	0.412	931	-0.011
0.417	3435	-0.028	0.417	1844	-0.020	0.417	934	-0.012
0.422	3445	-0.028	0.422	1852	-0.020	0.422	936	-0.012
0.427	3450	-0.029	0.427	1853	-0.020	0.427	940	-0.012
0.432	3453	-0.029	0.432	1857	-0.020	0.432	942	-0.012
0.437	3453	-0.030	0.437	1854	-0.021	0.437	944	-0.013
0.442	3463	-0.031	0.442	1852	-0.021	0.442	946	-0.013
0.447	3473	-0.032	0.447	1857	-0.021	0.447	951	-0.013
0.452	3481	-0.032	0.452	1868	-0.021	0.452	954	-0.014
0.457	3494	-0.032	0.457	1864	-0.021	0.457	955	-0.014
0.462	3507	-0.032	0.462	1869	-0.022	0.462	958	-0.014
0.467	3517	-0.033	0.467	1864	-0.022	0.467	960	-0.014
0.472	3530	-0.033	0.472	1860	-0.022	0.472	963	-0.014
0.477	3543	-0.033	0.477	1862	-0.023	0.477	968	-0.014
0.482	3548	-0.033	0.482	1864	-0.023	0.482	972	-0.015
0.487	3561	-0.034	0.487	1861	-0.024	0.487	972	-0.015
0.492	3576	-0.035	0.492	1862	-0.024	0.492	972	-0.015
0.497	3584	-0.036	0.497	1869	-0.024	0.497	972	-0.015
0.502	3592	-0.036	0.501	1871	-0.024	0.502	974	-0.015
0.507	3600	-0.036				0.507	980	-0.015
0.512	3610	-0.037				0.512	983	-0.016
0.517	3625	-0.037				0.515	982	-0.016
0.522	3636	-0.038						
0.527	3641	-0.038						
0.532	3646	-0.039						
0.537	3659	-0.040						
0.542	3672	-0.040						
0.547	3679	-0.041						
0.552	3682	-0.041						
0.557	3695	-0.041						
0.562	3705	-0.042						
0.567	3710	-0.043						
0.572	3718	-0.043						
0.577	3728	-0.044						
0.582	3749	-0.044						
0.587	3764	-0.045						
0.592	3783	-0.045						
0.597	3795	-0.046						
0.602	3806	-0.047						
0.607	3826	-0.047						
0.612	3842	-0.048						
0.617	3855	-0.048						
0.622	3870	-0.048						
0.627	3880	-0.049						
0.632	3878	-0.050						
0.634	3886	-0.051						

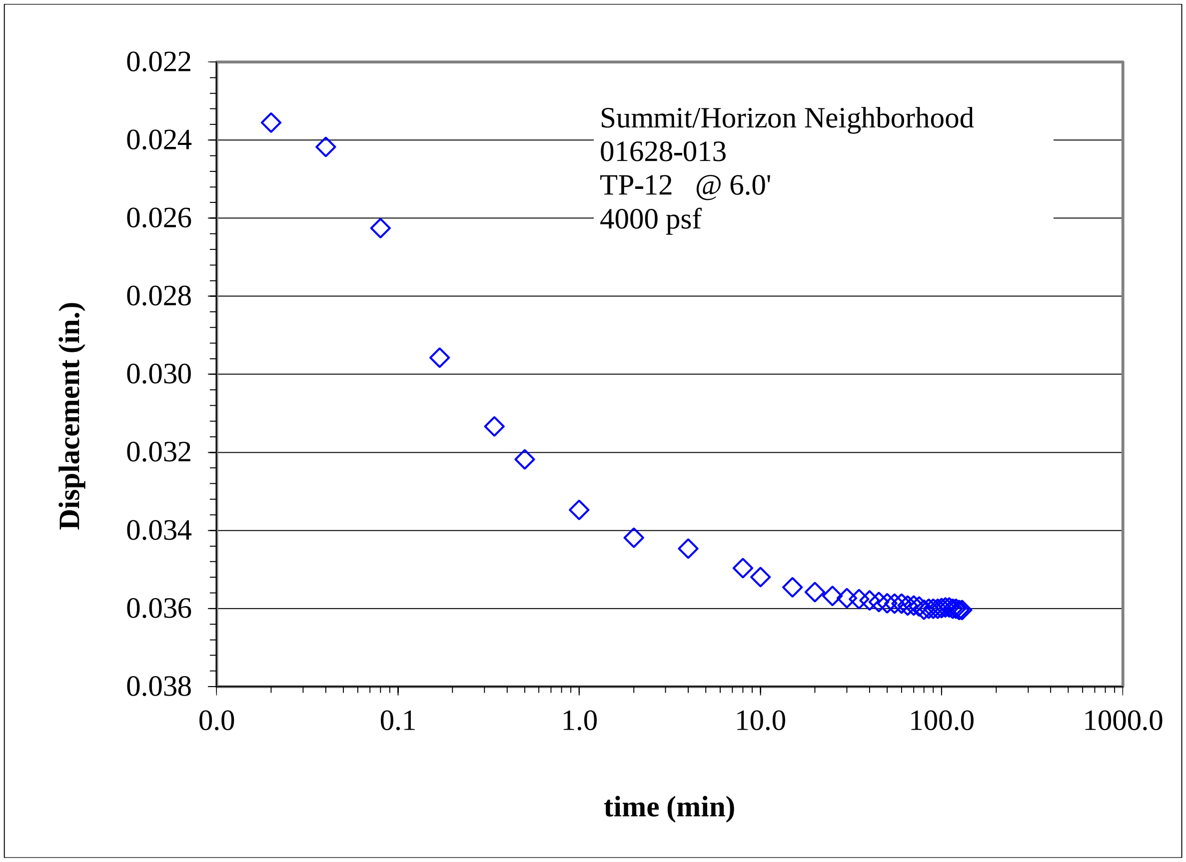
Direct Shear Test for Soils Under Drained Conditions

(ASTM D3080)

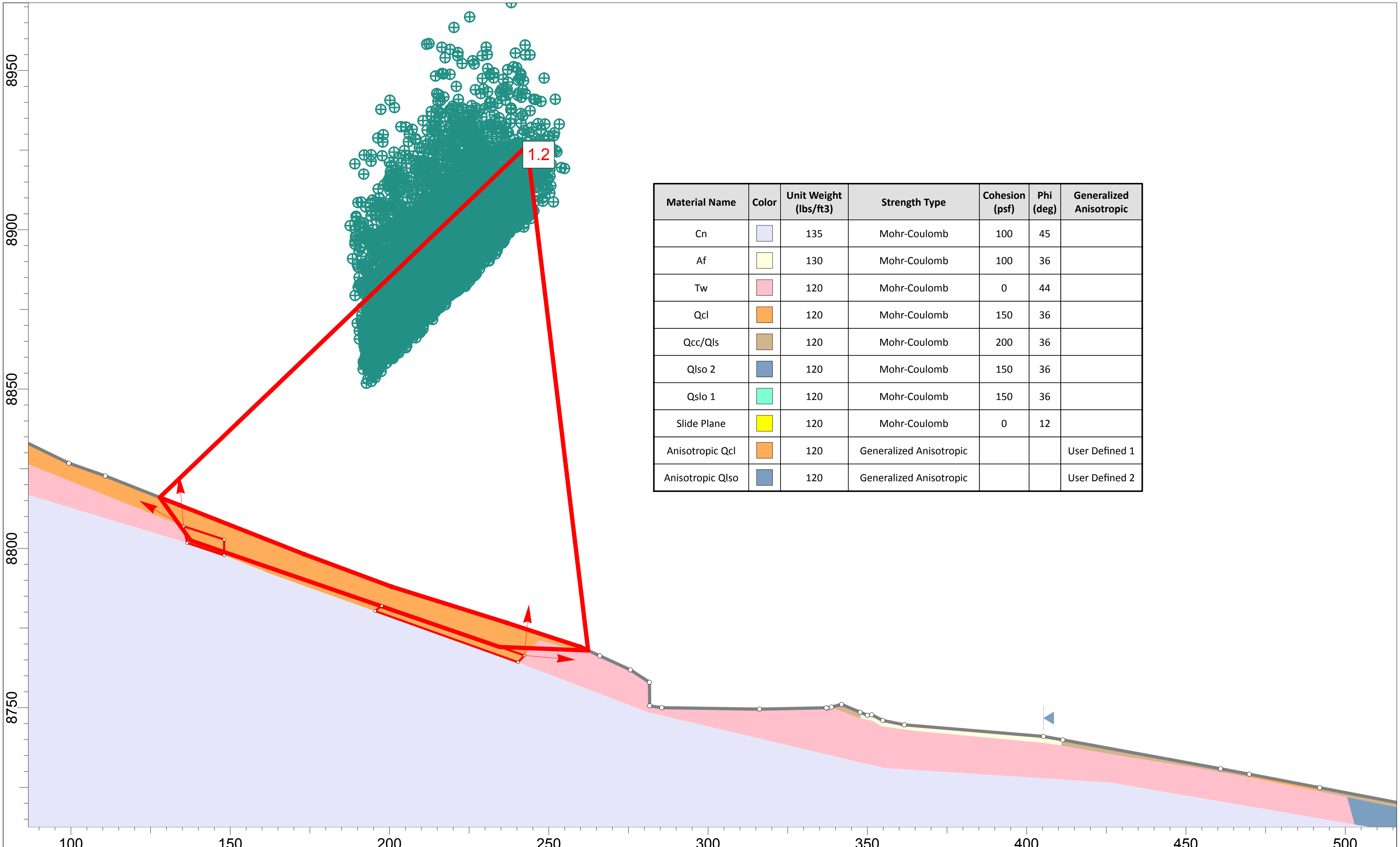
Project: Summit/Horizon Neighborhood
No: 01628-013
Location: Eden, UT

Boring No.: TP-12
Sample:
Depth: 6.0'





APPENDIX C



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	100	45	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qcc/Qls		120	Mohr-Coulomb	200	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Slide Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

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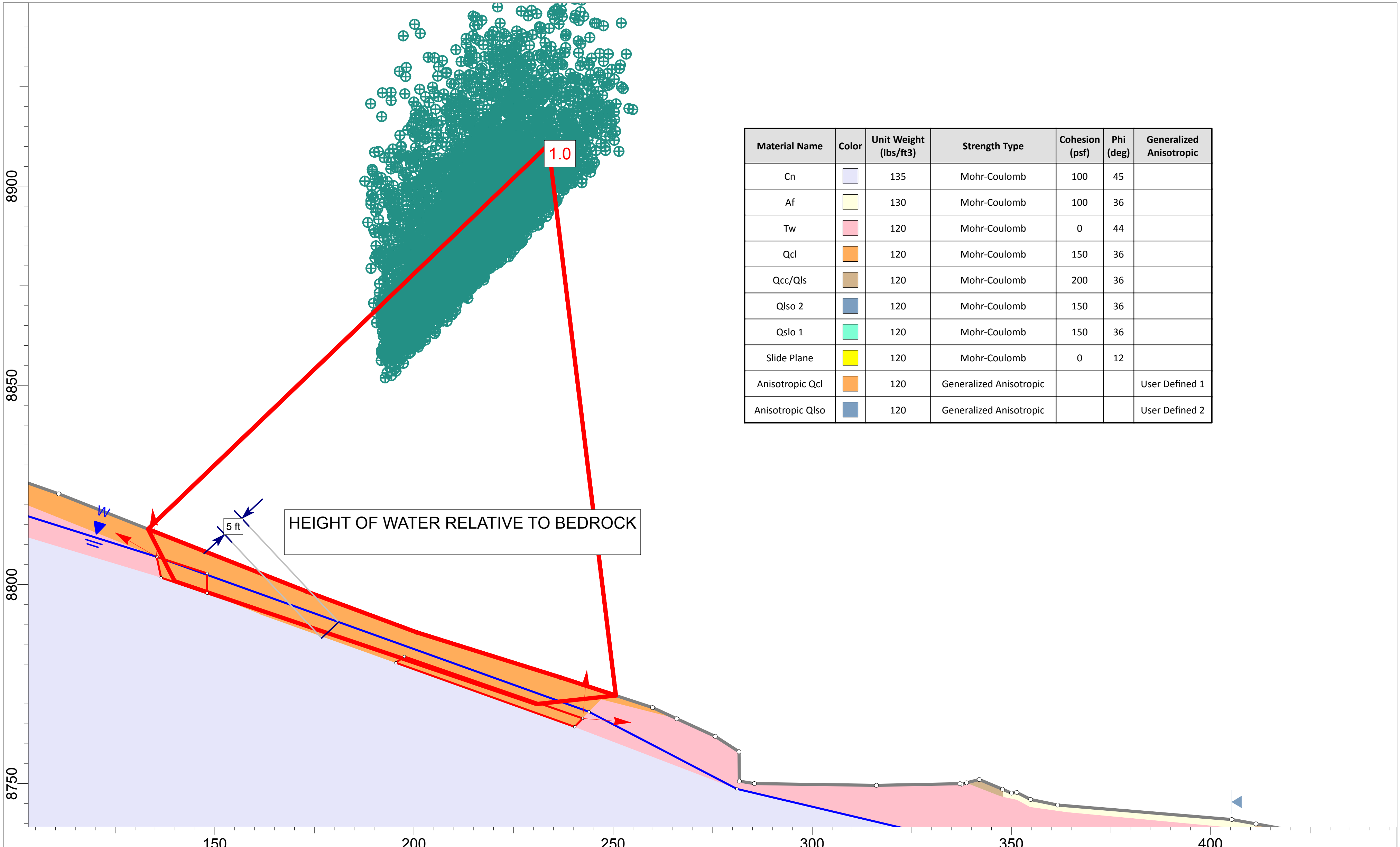
SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah
Analysis Description	Section A-A' - Static Analysis
Drawn By TQH	*Company* IGES
Date 7-27-2016	*File Name* A-A' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	100	45	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qcc/Qls		120	Mohr-Coulomb	200	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Slide Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

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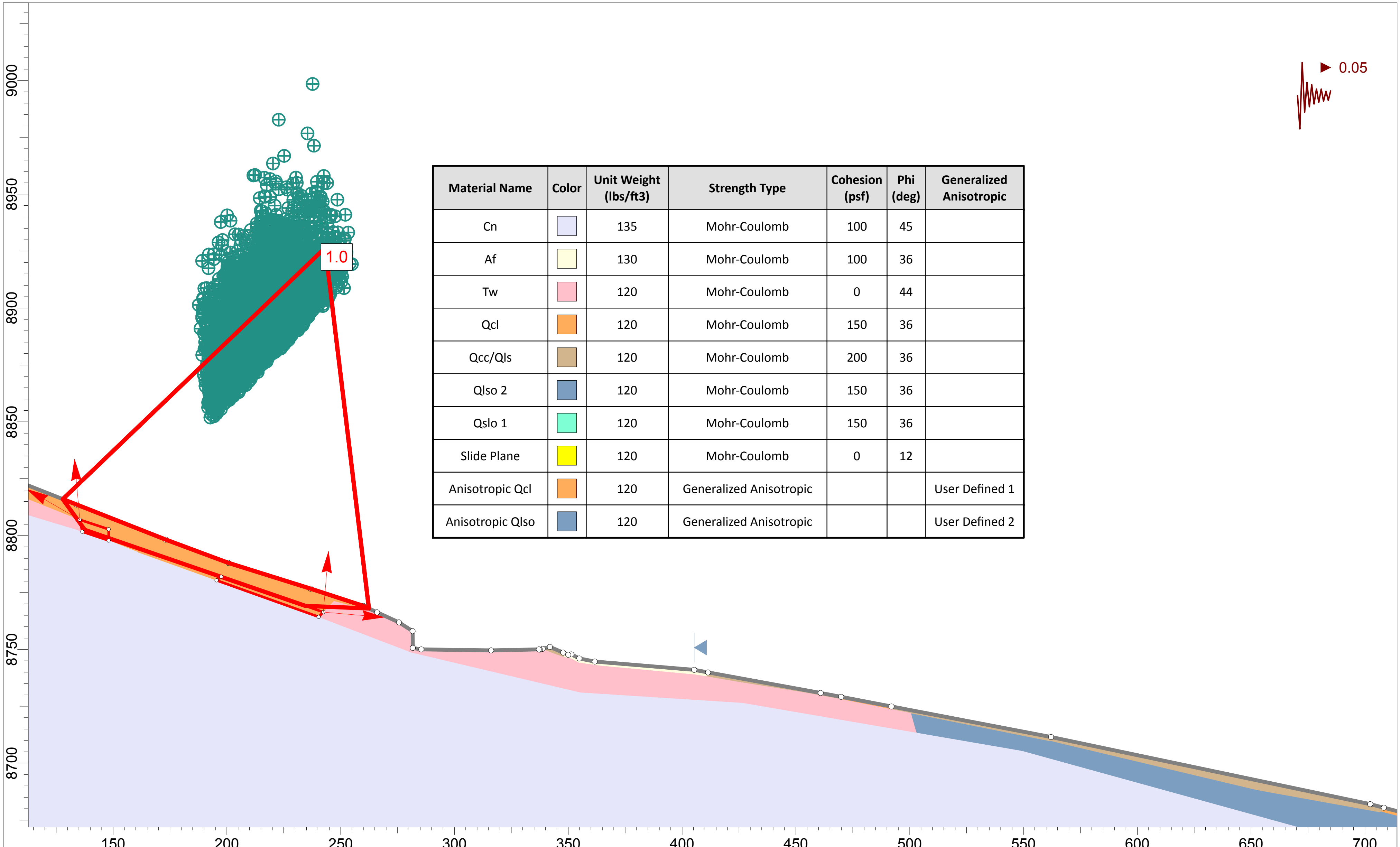
SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah		
Analysis Description	Section A-A' - Water Table Fluctuations		
Drawn By	TQH	*Company*	IGES
Date	7-27-2016	*File Name*	A-A' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	100	45	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qcc/Qls		120	Mohr-Coulomb	200	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Slide Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

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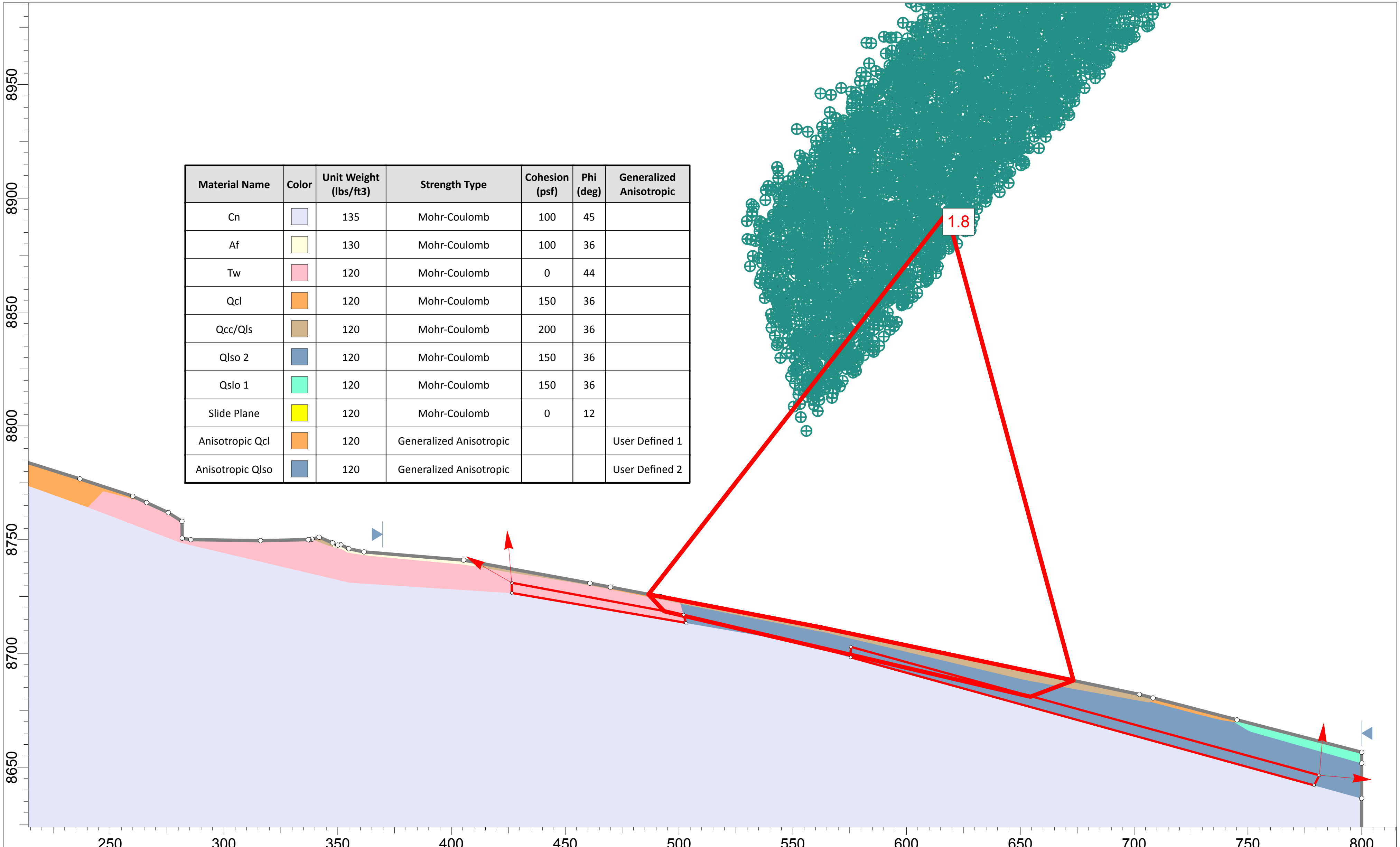
SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah		
Analysis Description	Section A-A' - Yield Acceleration		
Drawn By	TQH	*Company*	IGES
Date	7-27-2016	*File Name*	A-A' Seismic.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	100	45	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qcc/Qls		120	Mohr-Coulomb	200	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Slide Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

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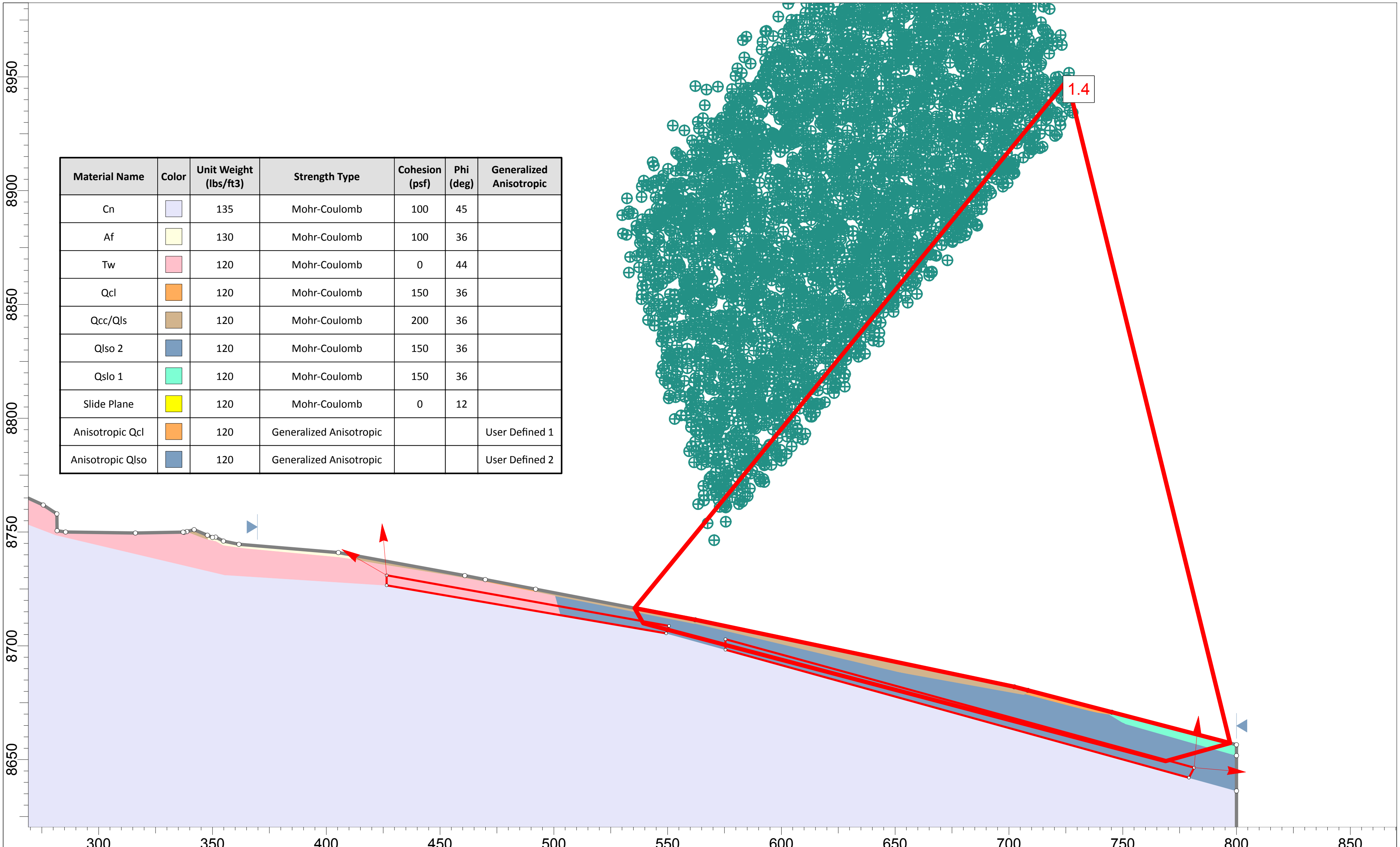
SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah
Analysis Description	Section A-A' - Downslope Static
Drawn By	TQH
Company	IGES
Date	7-27-2016
File Name	A-A' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	100	45	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qcc/Qls		120	Mohr-Coulomb	200	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Slide Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

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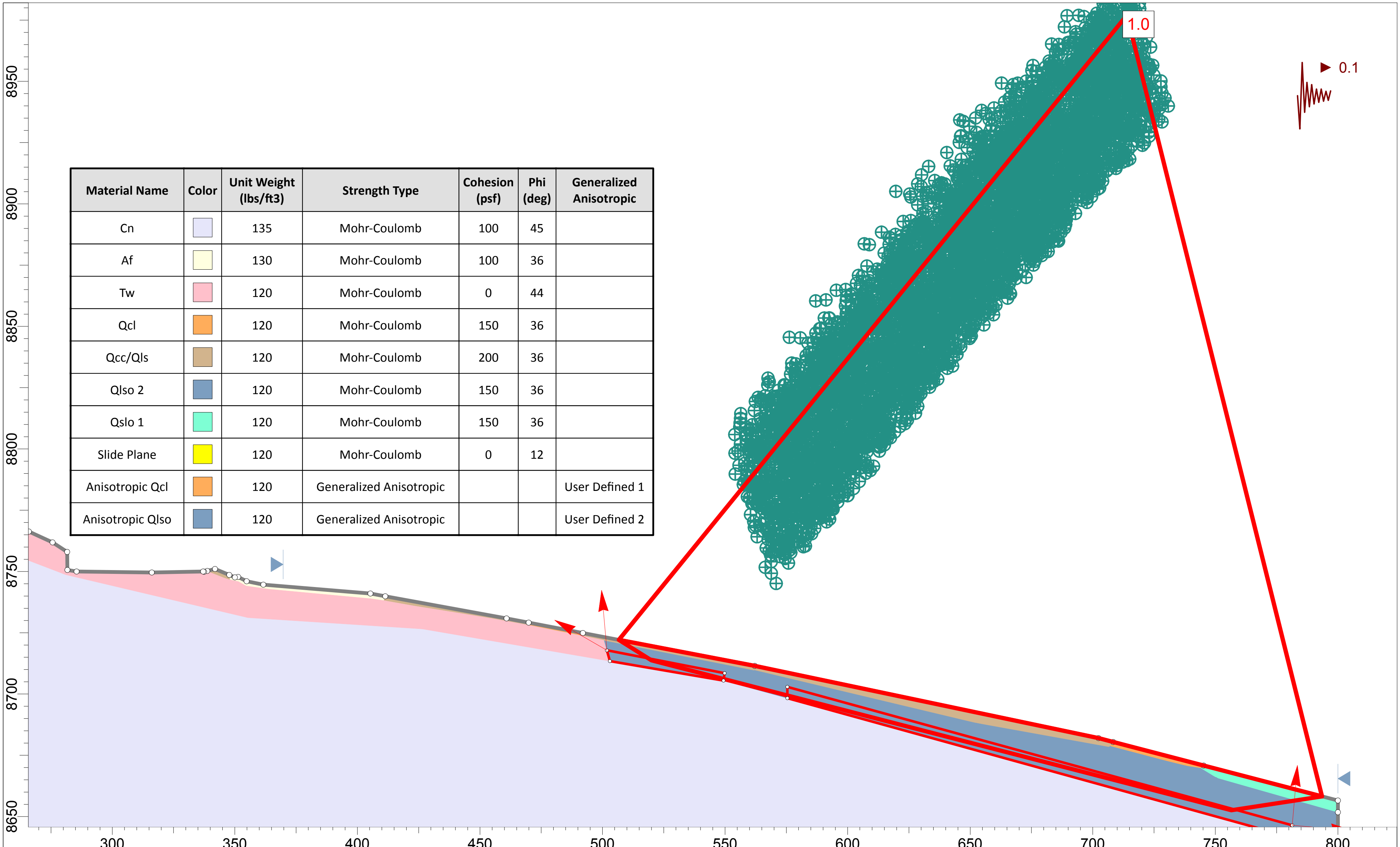
SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah		
Analysis Description	Section A-A' - Downslope Static		
Drawn By	TQH	*Company*	IGES
Date	7-27-2016	*File Name*	A-A' Static.slim

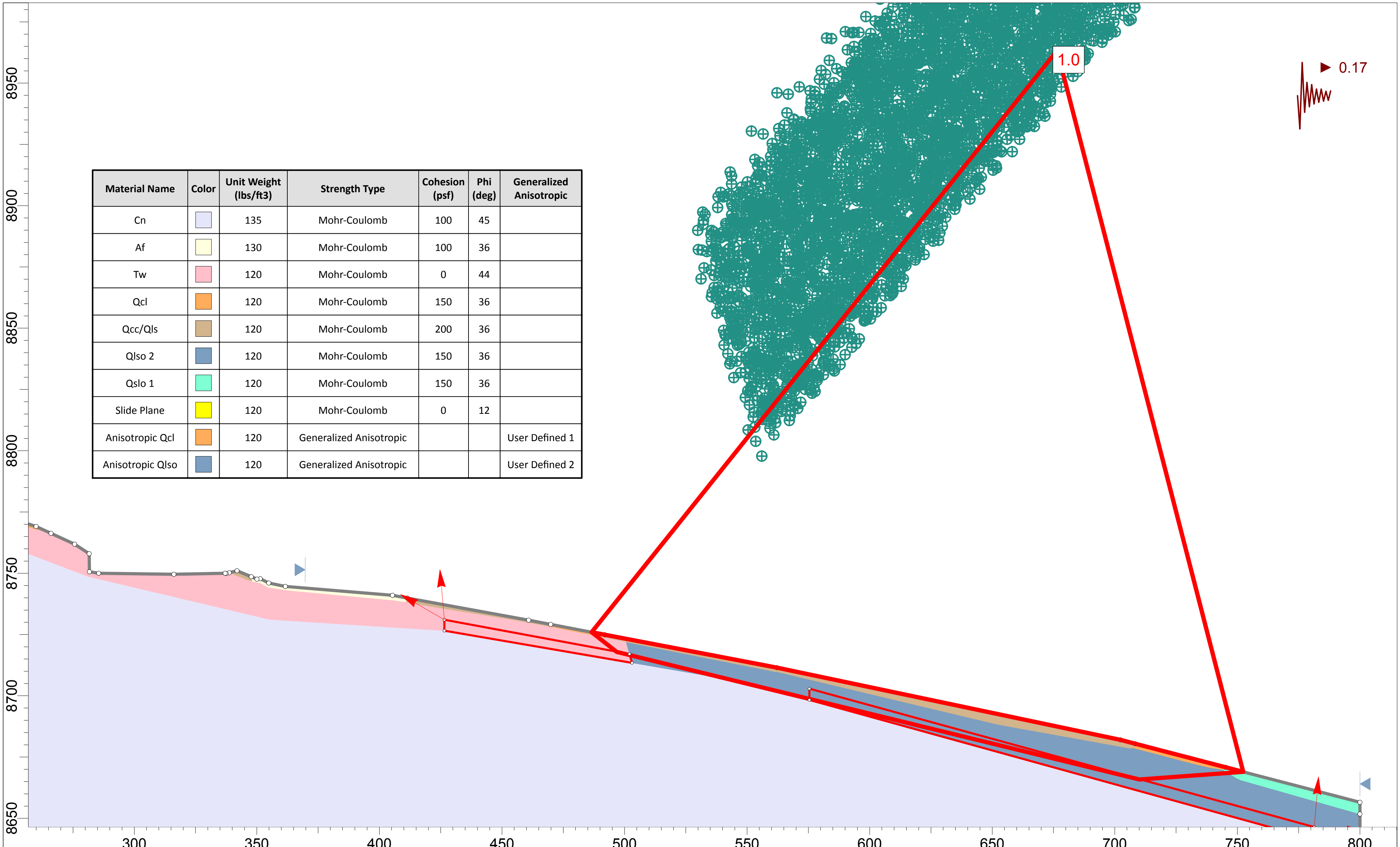


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	100	45	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qcc/Qls		120	Mohr-Coulomb	200	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Slide Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

IGES®

SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah		
Analysis Description	Section A-A' - Downslope Yield Acceleration		
Drawn By	TQH	*Company*	IGES
Date	7-27-2016	*File Name*	A-A' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	100	45	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qcc/Qls		120	Mohr-Coulomb	200	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Slide Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

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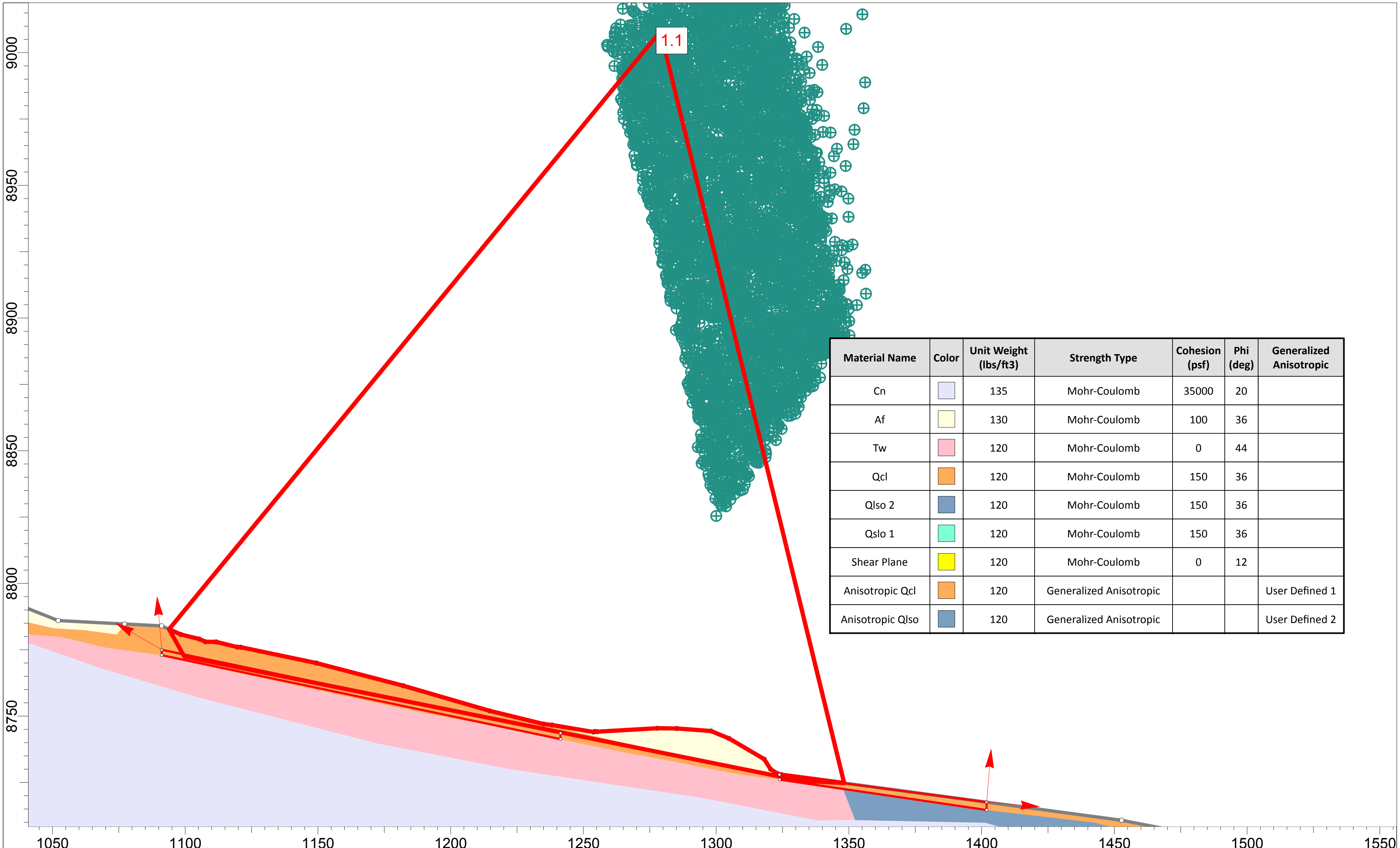
SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah
Analysis Description	Section A-A' - Downslope Yield Acceleration
Drawn By	TQH
Company	IGES
Date	7-27-2016
File Name	A-A' Static.slim

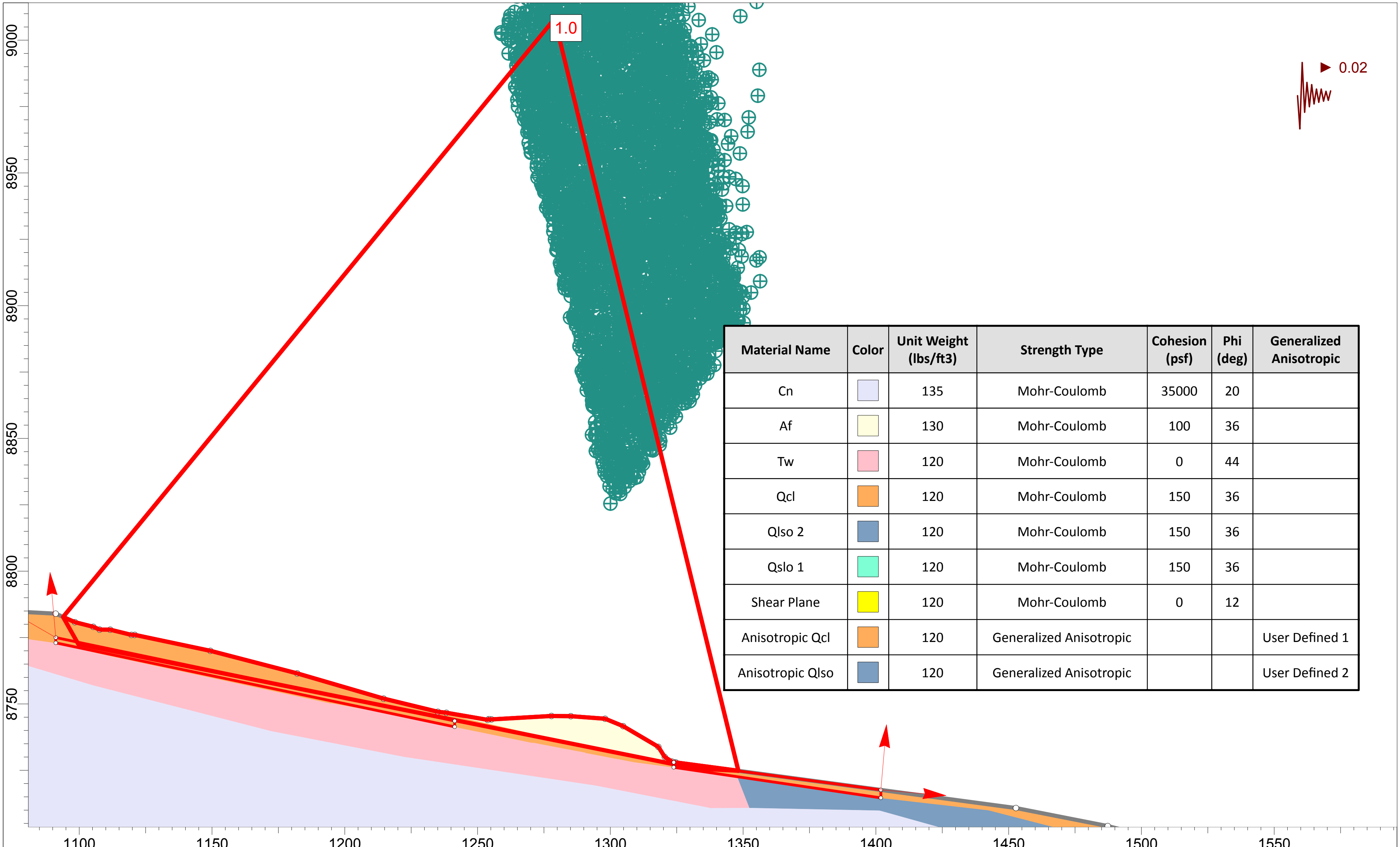


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	35000	20	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Shear Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

IGES®

SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah
Analysis Description	Section B-B' - Static
Drawn By	TQH
Company	IGES
Date	7-27-2016
File Name	B-B' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	35000	20	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Shear Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

IGES®

SLIDEINTERPRET 6.024

Project
Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah

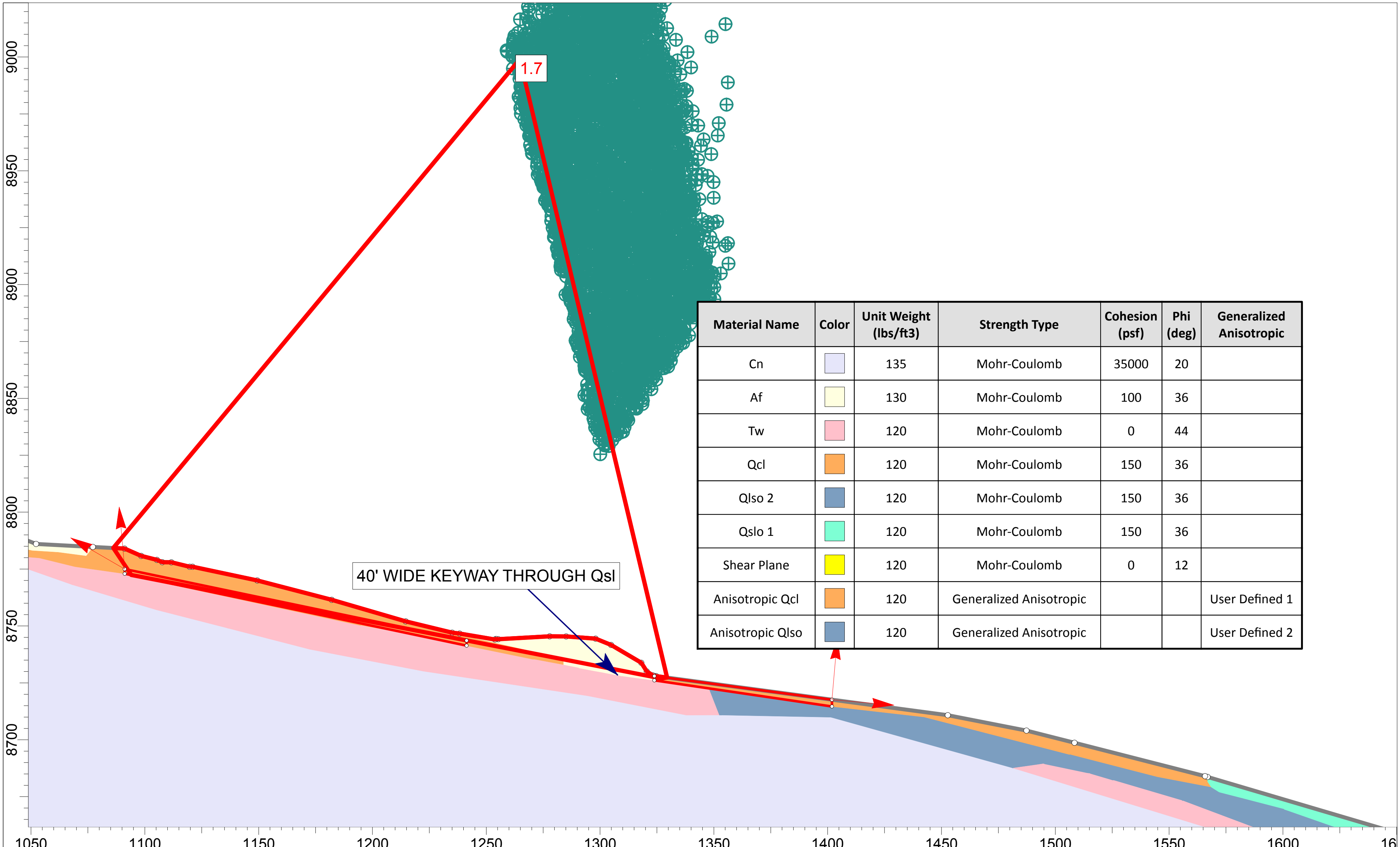
Analysis Description
Section B-B' - Seismic

Drawn By TQH

Company IGES

Date 7-27-2016

File Name B-B' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	35000	20	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Shear Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

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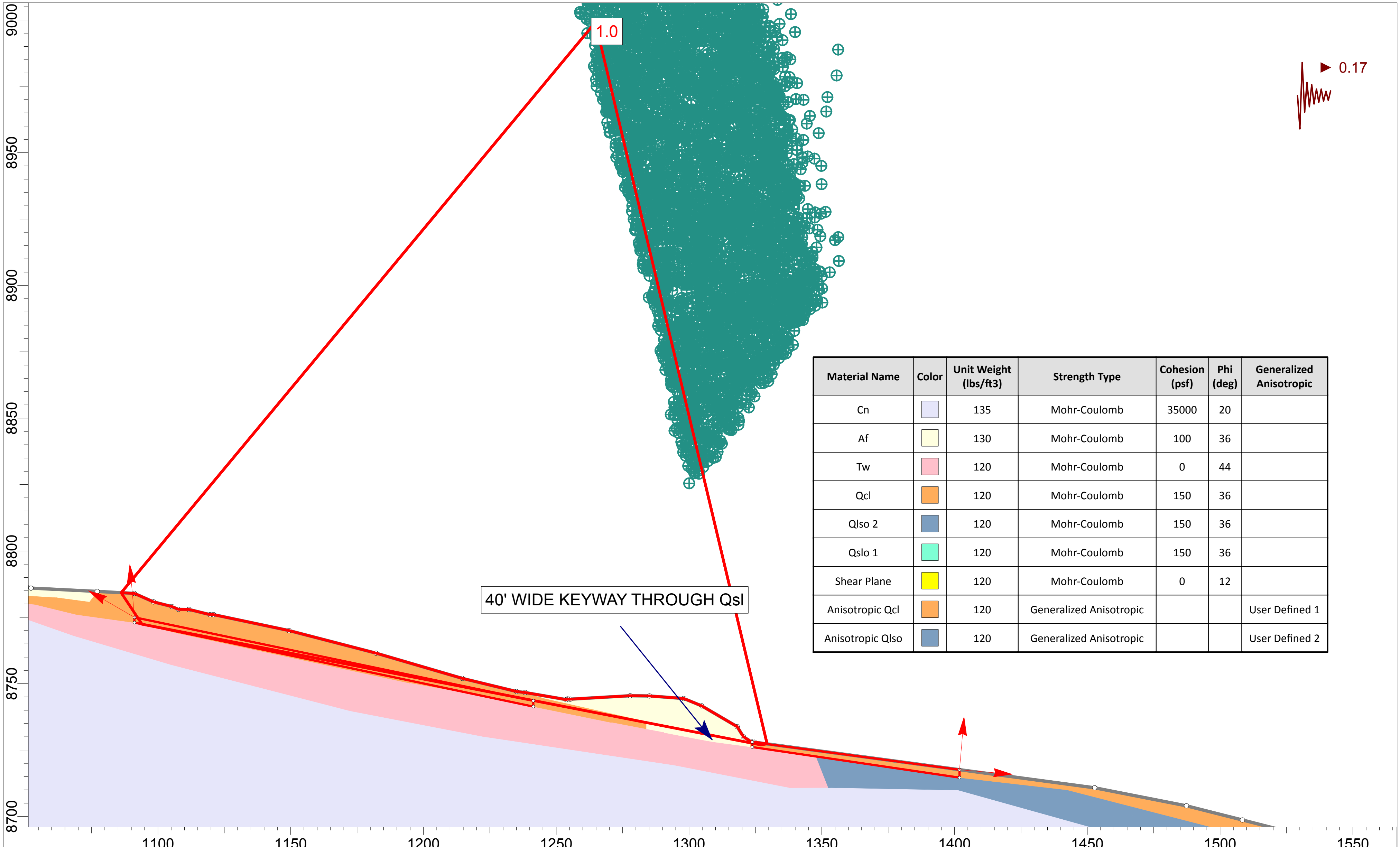
SLIDEINTERPRET 6.024

Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah
Analysis Description	Section B-B' - Static with Keyway
Drawn By TQH	*Company* IGES
Date 7-27-2016	*File Name* B-B' Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Generalized Anisotropic
Cn		135	Mohr-Coulomb	35000	20	
Af		130	Mohr-Coulomb	100	36	
Tw		120	Mohr-Coulomb	0	44	
Qcl		120	Mohr-Coulomb	150	36	
Qlso 2		120	Mohr-Coulomb	150	36	
Qslo 1		120	Mohr-Coulomb	150	36	
Shear Plane		120	Mohr-Coulomb	0	12	
Anisotropic Qcl		120	Generalized Anisotropic			User Defined 1
Anisotropic Qlso		120	Generalized Anisotropic			User Defined 2

IGES®

SLIDEINTERPRET 6.024

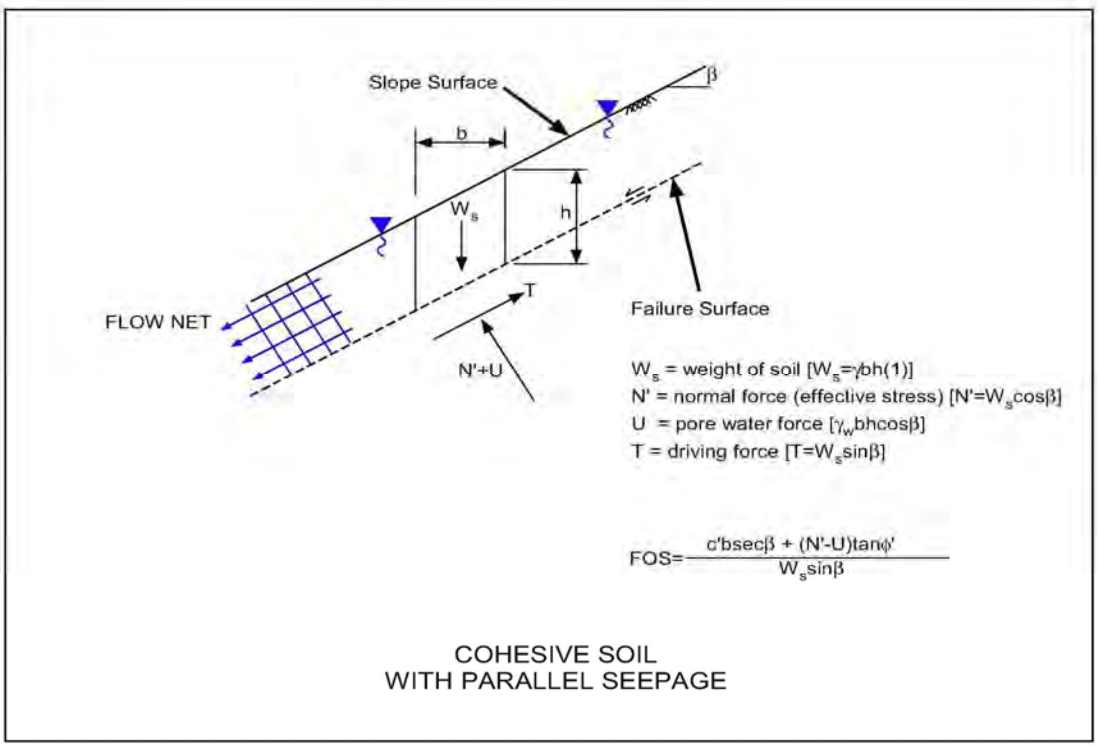
Project	Geotechnical and Geological Hazard Investigation - Summit Horizon Neighborhood, Utah		
Analysis Description	Section B-B' - Yield Acceleration		
Drawn By	TQH	*Company*	IGES
Date	7-27-2016	*File Name*	B-B' Seismic.slim

Horizon Neighbourhood
01628-013
7/27/2016

c'	150	psf	Effective Cohesion
ϕ'	36	deg	Effective Friction Angle
Y_{sat}	135	pcf	Saturated Unit Weight of Soil
Y_w	62.4	pcf	Unit weight of water
h	4	ft	Depth to shear surface
β	20.3	deg	Slope Gradient (2.7H:1V)

FS	1.91	
		Input Variable
		Calculated Value

This model assumes c>0 and the face of the slope is saturated to depth h



APPENDIX D

USGS Design Maps Detailed Report

2012/2015 International Building Code (41.368°N, 111.7608°W)

Site Class B – "Rock", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] $S_S = 0.829$ g

From Figure 1613.3.1(2) [2] $S_1 = 0.276$ g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class B, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = B and S_S = 0.829 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = B and S_1 = 0.276 g, F_v = 1.000

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 0.829 = 0.829$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.000 \times 0.276 = 0.276$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 0.829 = 0.553$ g

Equation (16-40): $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.276 = 0.184$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.553 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.184 g, Seismic Design Category = C

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

USGS Design Maps Summary Report

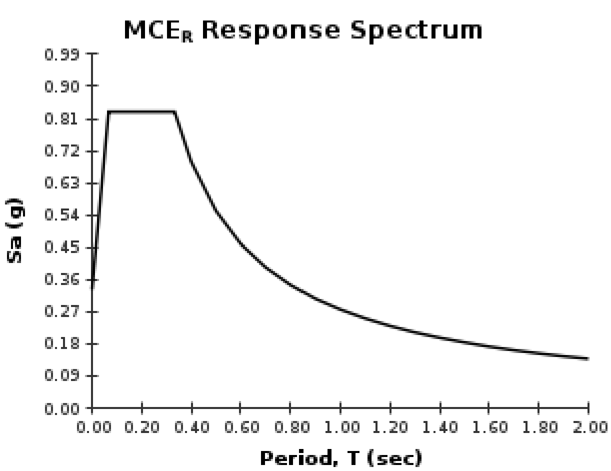
User–Specified Input

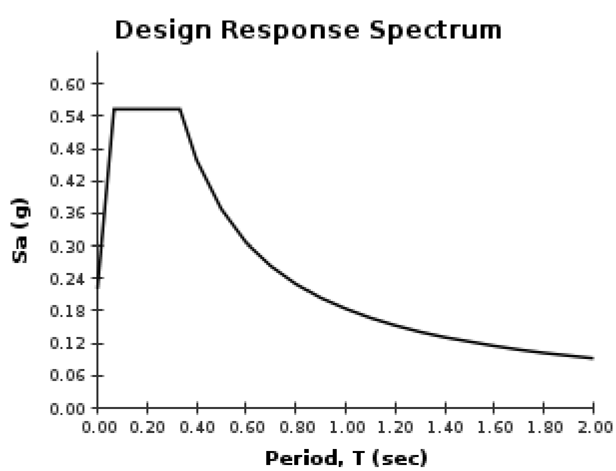
Report Title	Horizon Neighborhood Wed July 20, 2016 23:45:31 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	41.368°N, 111.7608°W
Site Soil Classification	Site Class B – “Rock”
Risk Category	I/II/III

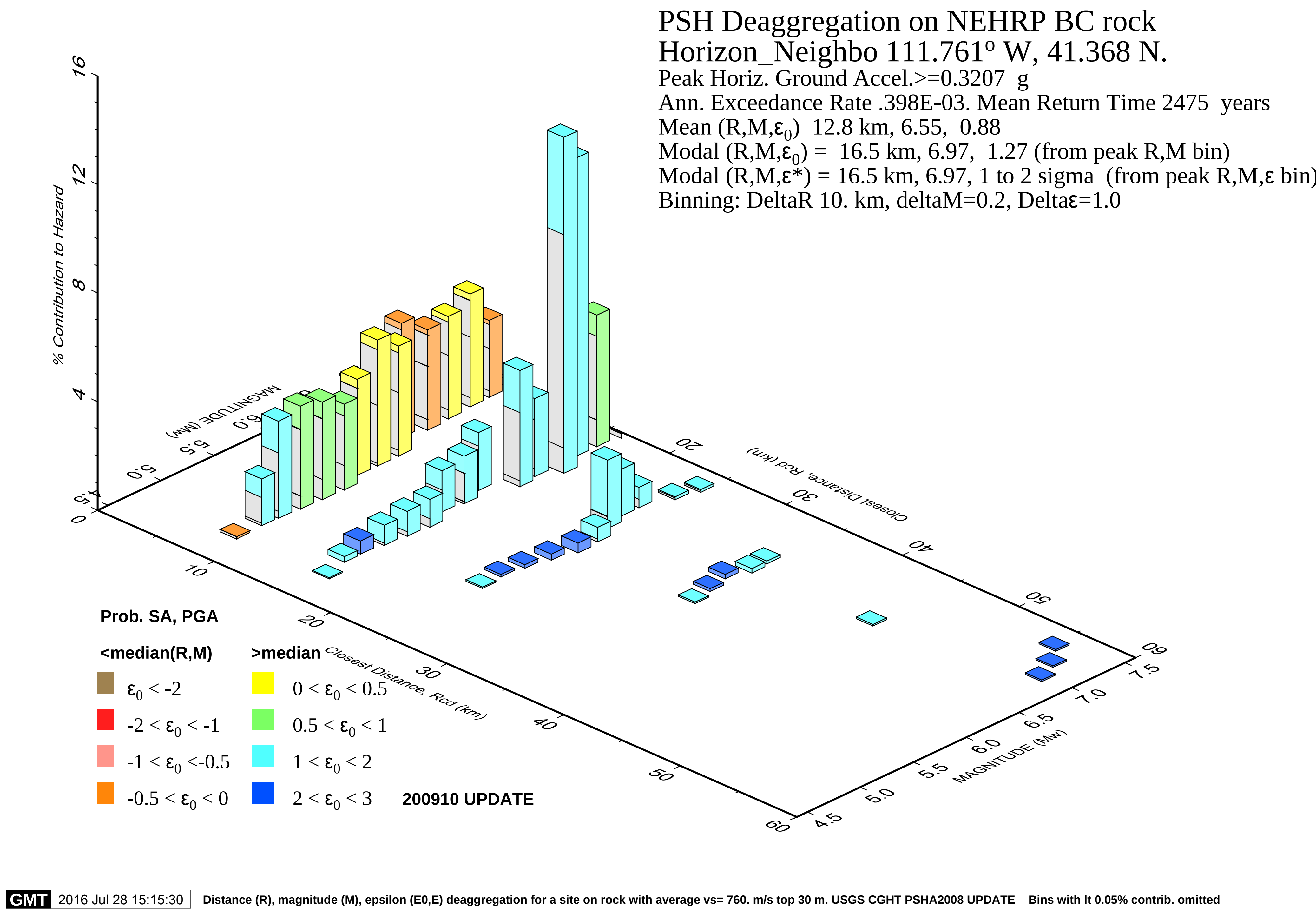
USGS–Provided Output

S_S =	0.829 g	S_{MS} =	0.829 g	S_{DS} =	0.553 g
S_1 =	0.276 g	S_{M1} =	0.276 g	S_{D1} =	0.184 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.







PSH Deaggregation on NEHRP BC rock
Horizon_Neighbo 111.761° W, 41.368 N.
Peak Horiz. Ground Accel.>=0.3207 g
Ann. Exceedance Rate .398E-03. Mean Return Time 2475 years
Mean (R,M,ε0) 12.8 km, 6.55, 0.88
Modal (R,M,ε0) = 16.5 km, 6.97, 1.27 (from peak R,M bin)
Modal (R,M,ε*) = 16.5 km, 6.97, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
0
4
8
12
16
MAGNITUDE (Mw)
4.5
5.0
5.5
6.0
6.5
7.0
7.5
Closest Distance, Rcd (km)
10
20
30
40
50
60
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2016 Jul 28 15:15:30
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

APPENDIX E

IGES

Intermountain GeoEnvironmental Services, Inc.

Project No. 01628-013

Horizon Neighbourhood

Date ______ by ______

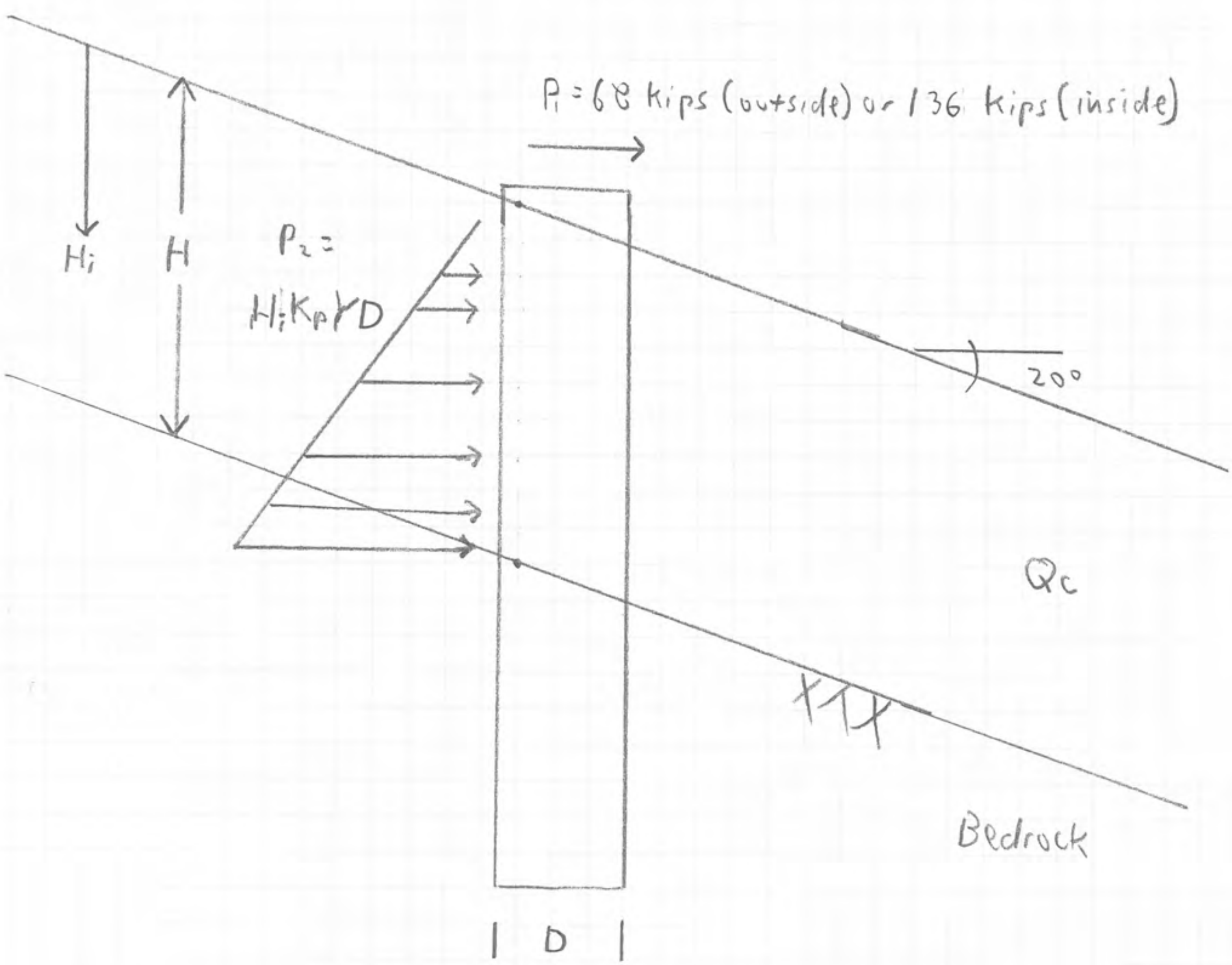
Ckd by ______ on ______

γ = 120 pcf

K_p = 8.335

P_1 = point load at top

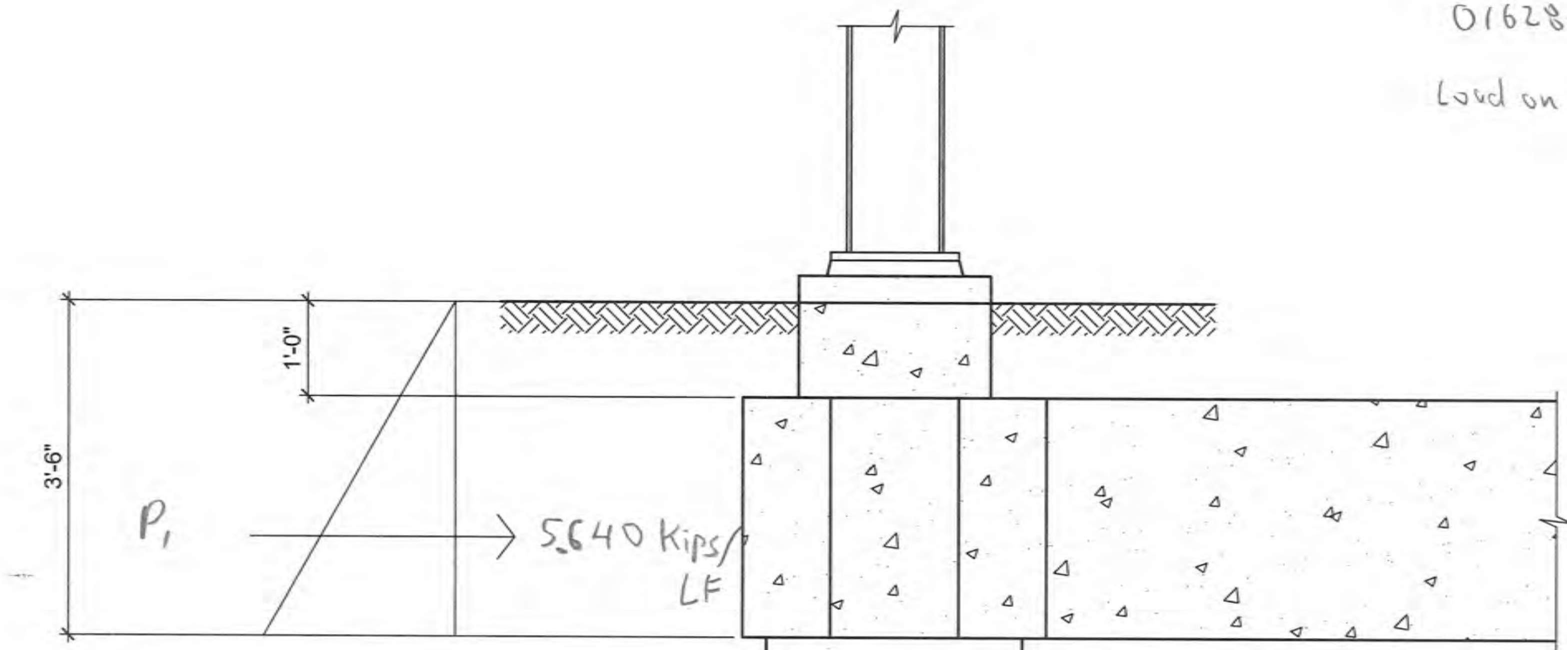
P_2 = distributed load



Basis for Drilled Pier Design

01628-013

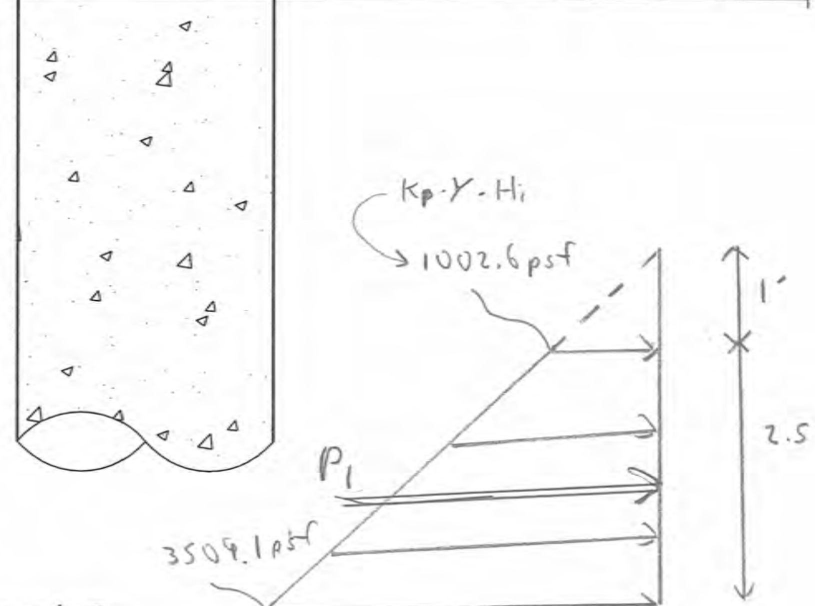
Load on grade beam



Load acting on grade beam

$\gamma = 120$ pcf

$K_p = 8.355$

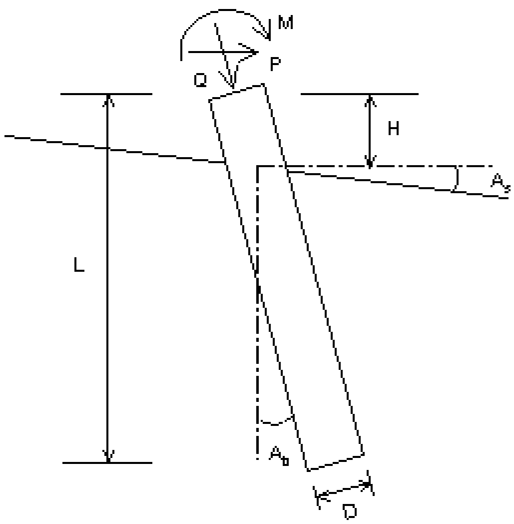


$P_1 = 1002.6\text{psf} \cdot 2.5' + (3509.1 - 1002.6)\text{psf}\ 2.5' \div 2$

$= 5640\ \text{lb/LF}$

LATERAL ANALYSIS

Figure E-1



Drilled Shaft (dia >24 in. or 61 cm)

Loads:

Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 100.0 -kp
Distributed Load:
Depth=0-ft Press.=0-kp/f2 Width=3.0-ft
Depth=16-ft Press.=8.064-kp/f2 Width=3.0-ft

Shear Load, P= 68.0 -kp
Moment, M= 0.0 -kp-f

Profile:

Pile Length, L= 24.0 -ft
Top Height, H= 16 -ft
Slope Angle, As= 20.0
Batter Angle, Ab= 0.0
Free Head Condition

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	165.0	26.0	626.00	2956.0	0.03	60
3	165.0	35.0	1044.00	2956.0	0.03	60

Pile Data:

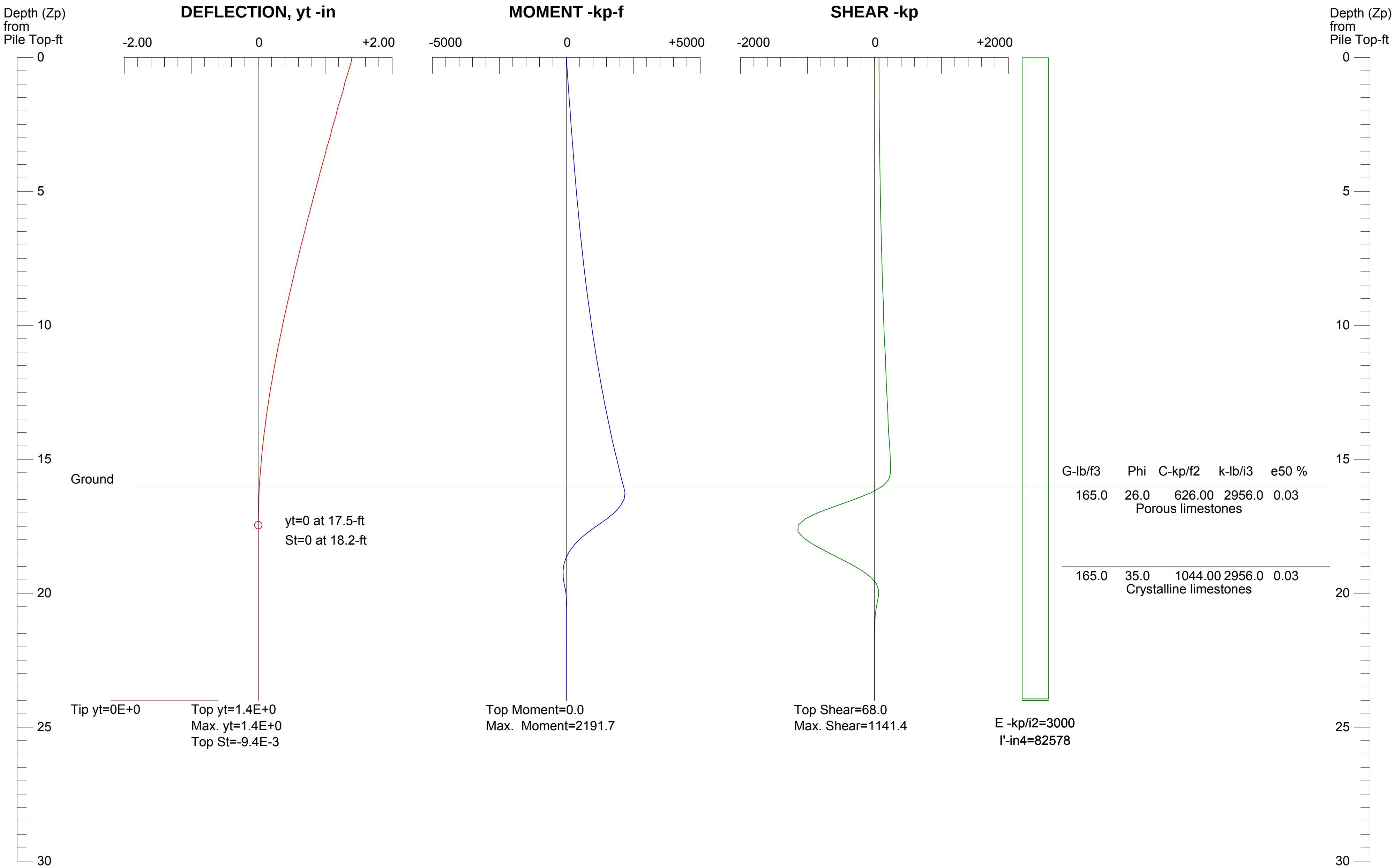
Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	36	1116.9	113.1	82577.7	3000	1.087
24.0						

Single Pile Lateral Analysis:
Top Deflection, yt= 1.40000-in
Max. Moment, M= 2191.67-kp-f
Top Deflection Slope, St= -0.00940
OK! Top Deflection, 1.4000-in is less than the Allowable Deflection= 2.00-in

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.
The Max. Moment calculated by program is an internal force from the applied load conditions. Structural engineer has to check whether the pile has enough capacity to resist the moment with adequate factor of safety. If not, the pile may fail under the load conditions.

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=2, Kbc=1



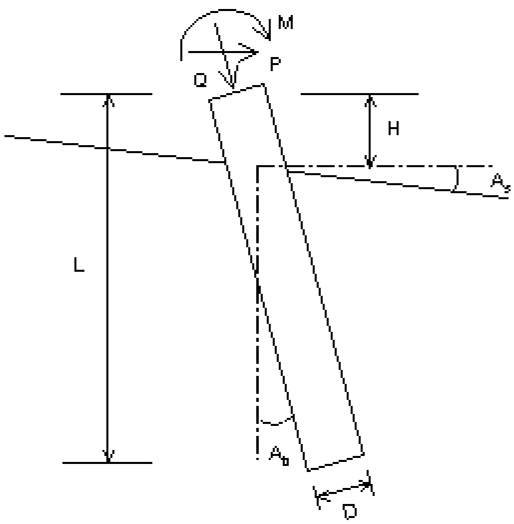
IGES®

Summit/Horizon Neighborhood
36" dia. Pier 16H Outside Pier

Figure E-2

```
****************************************************************
                 ALLPILE 7
      LATERAL ANALYSIS SUMMARY OUTPUT
       Copyright by CivilTech Software
           www.civiltechsoftware.com
****************************************************************
   Distributed Load:
        Depth   Press.          Width
        -ft     -kp/f2          -ft
        0       0               3.0
        16      8.064           3.0

FACTORS AND CONDITIONS:
        Load Factor for Vertical Loads: 1.0
        Load Factor for Lateral Loads: 1.0
        Loads Supported by Pile Cap: 0 %
        Shear Condition: Static

SINGLE PILE:
        (with Load Factor)
        Vertical Load= 100.00 -kp
        Shear= 68.00 -kp
        Moment= 0.00 -kp-f

Results:
        Top Deflection, yt= 1.40000-in
        Max. Moment, M= 2191.67-kp-f
        Top Deflection Slope, St= -0.00940

        Top Deflection, 1.4000-in, OK with the Allowable Deflection= 2.00-in

Note:  If the program cannot find a result or the result exceeds the upper limit.
The result will be displayed as 99999.
____________________________________

Notes:
 Q - Vertical Load at pile top
 P - Lateral Shear Load at pile top
 M - Moment at pile top
 Xtop - Pile top total settlement
 yt - Pile top deflection
 St - Pile top deflection slope (deflection/unit length)


The Max. Moment calculated by program is an internal moment of shaft due to the
loading. Egineers
have to check whether the pile has enough moment capacity to resist the Max. Moment
with adequate
factor of safety.  If not, the pile may be damaged under the loading.
1       1       1       1       1
```

LATERAL ANALYSIS



Drilled Shaft (dia >24 in. or 61 cm)

Loads:
Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 100.0 -kp
Distributed Load:
Depth=0-ft Press.=0-kp/f2 Width=3.5-ft
Depth=16-ft Press.=8.064-kp/f2 Width=3.5-ft

Shear Load, P= 136.0 -kp
Moment, M= 0.0 -kp-f

Profile:
Pile Length, L= 24.0 -ft
Top Height, H= 16 -ft
Slope Angle, As= 20.0
Batter Angle, Ab= 0.0
Free Head Condition

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	165.0	26.0	626.00	2956.0	0.03	60
3	165.0	35.0	1044.00	2956.0	0.03	60

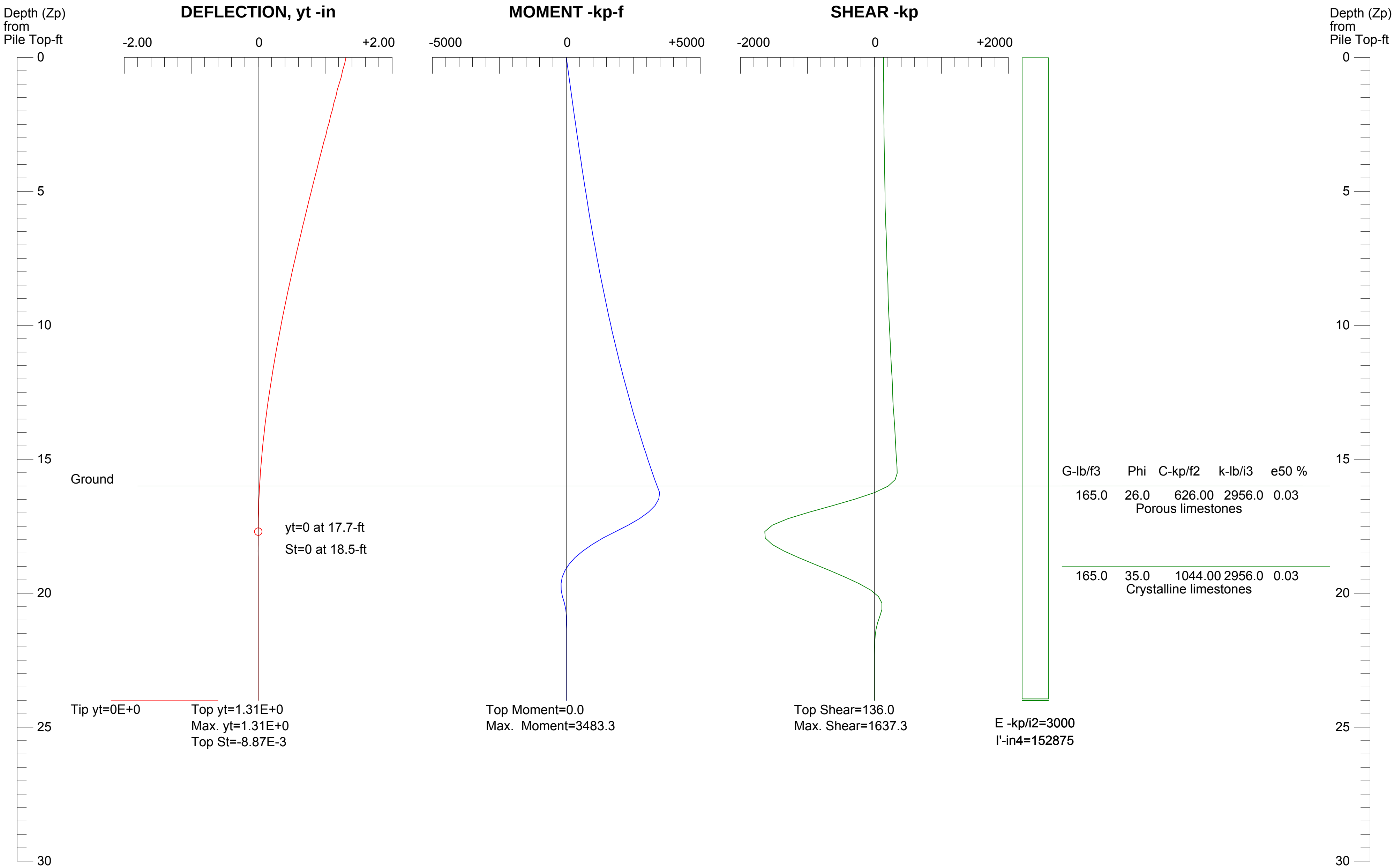
Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	42	1484.5	131.9	152874.8	3000	1.470
24.0						

Single Pile Lateral Analysis:
Top Deflection, yt= 1.31000-in
Max. Moment, M= 3483.33-kp-f
Top Deflection Slope, St= -0.00887
OK! Top Deflection, 1.3100-in is less than the Allowable Deflection= 2.00-in

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.
The Max. Moment calculated by program is an internal force from the applied load conditions. Structural engineer has to check whether the pile has enough capacity to resist the moment with adequate factor of safety. If not, the pile may fail under the load conditions.

PILE DEFLECTION & FORCE vs DEPTH
Single Pile, Khead=2, Kbc=1



IGES

Summit/Horizon Neighborhood
42" dia. Pier 16H Inside Pier

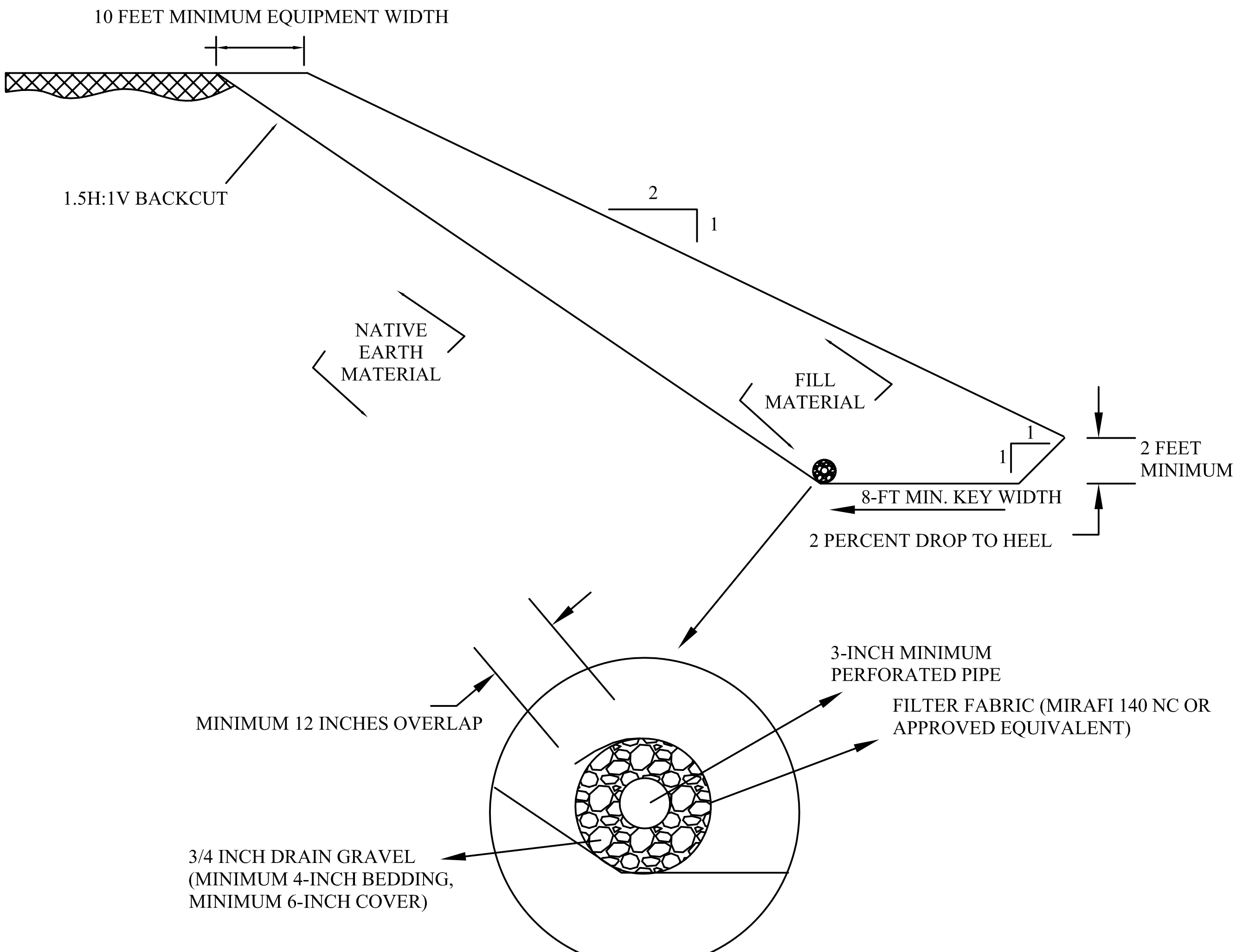
Figure E-5

```
**************************************************************
                 ALLPILE 7
      LATERAL ANALYSIS SUMMARY OUTPUT
       Copyright by CivilTech Software
           www.civiltechsoftware.com
**************************************************************
   Distributed Load:
        Depth   Press.          Width
        -ft     -kp/f2          -ft
        0       0               3.5
        16      8.064           3.5

FACTORS AND CONDITIONS:
        Load Factor for Vertical Loads: 1.0
        Load Factor for Lateral Loads: 1.0
        Loads Supported by Pile Cap: 0 %
        Shear Condition: Static

SINGLE PILE:
        (with Load Factor)
        Vertical Load= 100.00 -kp
        Shear= 136.00 -kp
        Moment= 0.00 -kp-f

Results:
        Top Deflection, yt= 1.31000-in
        Max. Moment, M= 3483.33-kp-f
        Top Deflection Slope, St= -0.00887

        Top Deflection, 1.3100-in, OK with the Allowable Deflection= 2.00-in

Note:  If the program cannot find a result or the result exceeds the upper limit.
The result will be displayed as 99999.
____________________________________

Notes:
 Q - Vertical Load at pile top
 P - Lateral Shear Load at pile top
 M - Moment at pile top
 Xtop - Pile top total settlement
 yt - Pile top deflection
 St - Pile top deflection slope (deflection/unit length)


The Max. Moment calculated by program is an internal moment of shaft due to the
loading. Egineers
have to check whether the pile has enough moment capacity to resist the Max. Moment
with adequate
factor of safety.  If not, the pile may be damaged under the loading.
1       1       1       1       1
```

APPENDIX F

FILL SLOPE DETAIL

NOTE: FOR THE LODGE STRUCTURE, MIN. KEYWAY DEPTH IS 5 FEET, MIN. KEYWAY LENGTH IS 25 FEET.



SUBDRAIN DETAIL

dap 4/4/08

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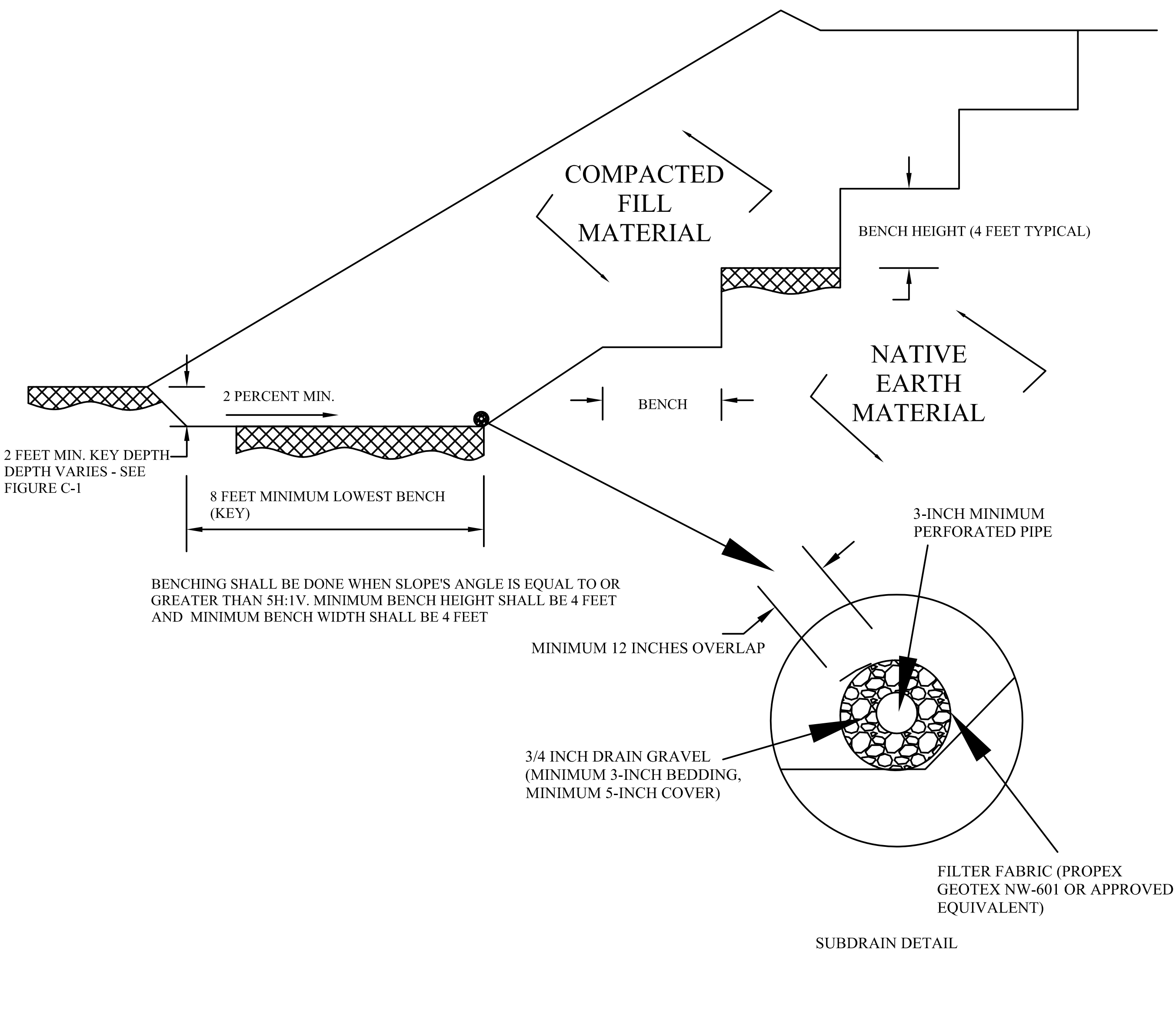
No.	Revision/Issue	Date

Geotechnical & Geologic Hazard Investigation
Horizon Neighbourhood Development
Summit at Powder Mountain Resort
Weber County, Utah
Project Number 01628-013

Drawing Notes

Figure F-1

KEYING AND BENCHING DETAIL



BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5H:1V. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM BENCH WIDTH SHALL BE 4 FEET

dap 4/4/08

IGES

No. | Revision/Issue | Date

Geotechnical & Geologic Hazard Investigation
Horizon Neighbourhood Development
Summit at Powder Mountain Resort
Weber County, Utah
Project Number 01628-013

Drawing Notes

Figure F-2